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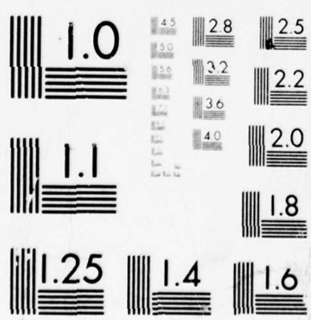

TECHNICAL REPORT H-77-15

IMPERIAL BEACH, CALIFORNIA DESIGN OF STRUCTURES FOR BEACH EROSION CONTROL

Hydraulic Model Investigation

by

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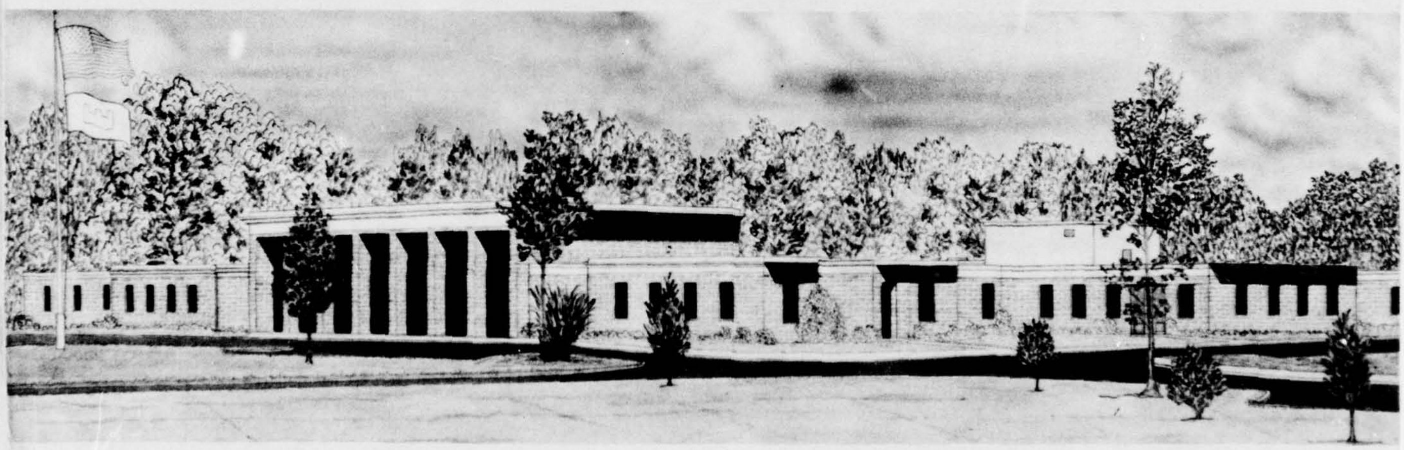
August 1977
Final Report



Prepared for U. S. Army Engineer District, Los Angeles
Los Angeles, California 90053

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Technical Report H-77-15	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER 9
4. TITLE (and Subtitle) 6 IMPERIAL BEACH, CALIFORNIA, DESIGN OF STRUCTURES FOR BEACH EROSION CONTROL; Hydraulic Model Investigation,		5. TYPE OF REPORT & PERIOD COVERED Final report, Sep 75 - Feb 77
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) 10 Charles R. Curren Claude E. Chatham, Jr		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Army Engineer Waterways Experiment Station Hydraulics Laboratory P. O. Box 631, Vicksburg, Mississippi 39180		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U. S. Army Engineer District, Los Angeles P. O. Box 2711 Los Angeles, California 90053	11	12. REPORT DATE August 1977 13. NUMBER OF PAGES 157 12
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office) 14 WES-TR-H-77-15		15. SECURITY CLASS. (of this report) Unclassified 161p. 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Beach erosion	Hydraulic models
Breakwaters	Imperial Beach, Calif.
Erosion control	Water wave experiments
Groins (structures)	

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A 1:75-scale (undistorted) hydraulic model, reproducing approximately 2.6 miles of shoreline and sufficient offshore area to permit generation of the required test waves, was used to investigate the arrangement and design of alternative proposed structures for prevention of erosion of the Imperial Beach shoreline. The proposed structures consisted of (a) continuous break-waters at the -15 ft and -10 ft contours, (b) segmented breakwaters at the -15 ft and -5 ft contours, (c) stepped breakwaters at the -10 ft and -5 ft

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20. ABSTRACT (Continued)

contours, (d) a system of five groins, and (e) a system of nine groins. A 115-ft-long wave generator, crushed coal tracer material, and an automated data acquisition and control system (ADACS) were used during model operation. It was concluded from model test results that:

a. Existing conditions are characterized by strong rip currents and longshore currents for most wave conditions with considerable onshore-offshore movement of sand.

b. The original improvement plan for Imperial Beach (i.e., the authorized five-groin plan) is ineffective in trapping tracer material and, in fact, contributes to the offshore movement.

c. The originally proposed breakwater plan (Plan 1) is not adequate for full protection of the beach. Raising the crown elevation of the Plan 1 structure to a sufficient height will make this plan effective from a functional standpoint; however, a substantial cost increase results due to greatly increased volume of rock required.

d. Of the plans tested, Plans 3A, 4A, 5A, 5B, and 7 provide adequate beach protection. Plans 4A and 5A require the least volume of rock but may pose some construction problems. Plan 5A would have the least impact on the longshore transport (due to its location at the -5.0 ft contour resulting in more natural sand bypassing around the structure) and therefore would be the most viable plan from the viewpoint of minimizing effect of the improvement on adjacent shores. The Plan 3A and 4A breakwaters would be submerged except for extremely low tide stages and, therefore, would probably be the most aesthetically pleasing.

PREFACE

A request for a model investigation of Imperial Beach, California, was initiated by the District Engineer, U. S. Army Engineer District, Los Angeles (SPL), in a letter to the Division Engineer, U. S. Army Engineer Division, South Pacific (SPD), dated 19 August 1974. Initial funds were authorized by SPL on 23 December 1974, with subsequent installments authorized through 23 March 1977.

The model study was conducted at the U. S. Army Engineer Waterways Experiment Station (WES) during the period September 1975 through February 1977 under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Dr. R. W. Whalin, Chief of the Wave Dynamics Division. The tests were conducted by Mr. C. R. Curren, Project Engineer, with the assistance of Messrs. L. A. Barnes, civil engineering technician, R. E. Ankeny, electronics technician, and K. A. Turner and C. W. Coe, computer specialists, under the supervision of Mr. C. E. Chatham, Chief of the Harbor Wave Action Branch. This report was prepared by Messrs. Curren and Chatham.

Prior to the model investigation, Mr. Chatham visited SPL and Imperial Beach to confer with representatives of SPD, SPL, and the Coastal Engineering Research Center (CERC) and to inspect the prototype site. During the course of the investigation, liason was maintained by means of conferences, telephone communications, and monthly progress reports.

The following personnel visited WES to observe model operation and participate in conferences during the course of the model study:

Mr. N. Parker, Office, Chief of Engineers (OCE)

Mr. J. Housley, OCE

Mr. S. Powell, OCE

Mr. M. L. Martin, OCE

BG R. M. Connell, Division Engineer, SPD

Mr. O. T. Magoon, SPD

Mr. A. E. Wanket, SPD

LTC R. H. Reinen, Deputy District Engineer, SPL

Mr. G. Fuquay, SPL

Mr. T. Nishihara, SPL

Mr. C. Fisher, SPL

Mr. D. Muslin, SPL

Mr. L. Morales, SPL

Mr. W. Collins, SPL

COL J. H. Cousins, Commander and Director, CERC

Mr. G. Watts, CERC

Mr. P. Stoa, CERC

Mr. T. Saville, CERC

Ms. K. Rees, CERC

Mr. J. Lesnik, CERC

Prof. R. L. Wiegel, Coastal Engineering Research Board (CERB)

Prof. R. G. Dean, CERB

Dean M. P. Obrien, CERB

HON B. Stites, Mayor, Imperial Beach, California

Mr. J. Shelver, City Manager, Imperial Beach, California

Mr. C. Johnson, North Central Division

Mr. R. J. Gorecki, Buffalo District

Mr. G. Armstrong, California Dept. of Oceanographic Development

Mr. A. Ruden, City Engineer, Oceanside, California

COL G. H. Hilt, CE, and COL John L. Cannon, CE, were Directors of WES during the conduct of this investigation and the preparation and publication of this report. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
square feet	0.09290304	square metres
square miles (U. S. statute)	2.589988	square kilometres
cubic yards	0.7645549	cubic metres
tons (2000 lb, mass)	907.1847	kilograms
feet per second	0.3048	metres per second
degrees (angle)	0.01745329	radians

IMPERIAL BEACH, CALIFORNIA

DESIGN OF STRUCTURES FOR BEACH EROSION CONTROL

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Imperial Beach is located on the Pacific Ocean about 3-1/2 miles* north of the Mexican Border and 11 miles south of San Diego, California (Figure 1). It is primarily a recreational beach, with a 1200-ft-long fishing pier situated in the approximate center of the study area. Two groins, 740 ft and 400 ft long, are located 2950 ft and 1625 ft north of the fishing pier, respectively. The sea floor is characterized by gently sloping contours with the Tijuana Shoal located south of the pier at the mouth of the Tijuana River.

The Problem

2. Inman[1] states that historically, the Tijuana River probably was the main source of sediment for Imperial Beach. However, since construction of the Morena and Barrett Dams in Cottonwood Creek and the Rodriguez Dam in the Rio de las Palmas, some sediment has become entrapped inland and never reaches the coast to replenish the supply of sand to the beaches. The lack of significant flood flows on the Tijuana River since 1944 has caused a severe shortage of sediment at the river mouth, resulting in a decreased quantity available for longshore transport to Imperial Beach; consequently, increased beach erosion has occurred.

3. The problem became acute during the winter of 1952-53 when wave erosion caused rapid shoreline recession and property damage. Winter

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4. All dimensions in this report are given in prototype units unless otherwise noted.

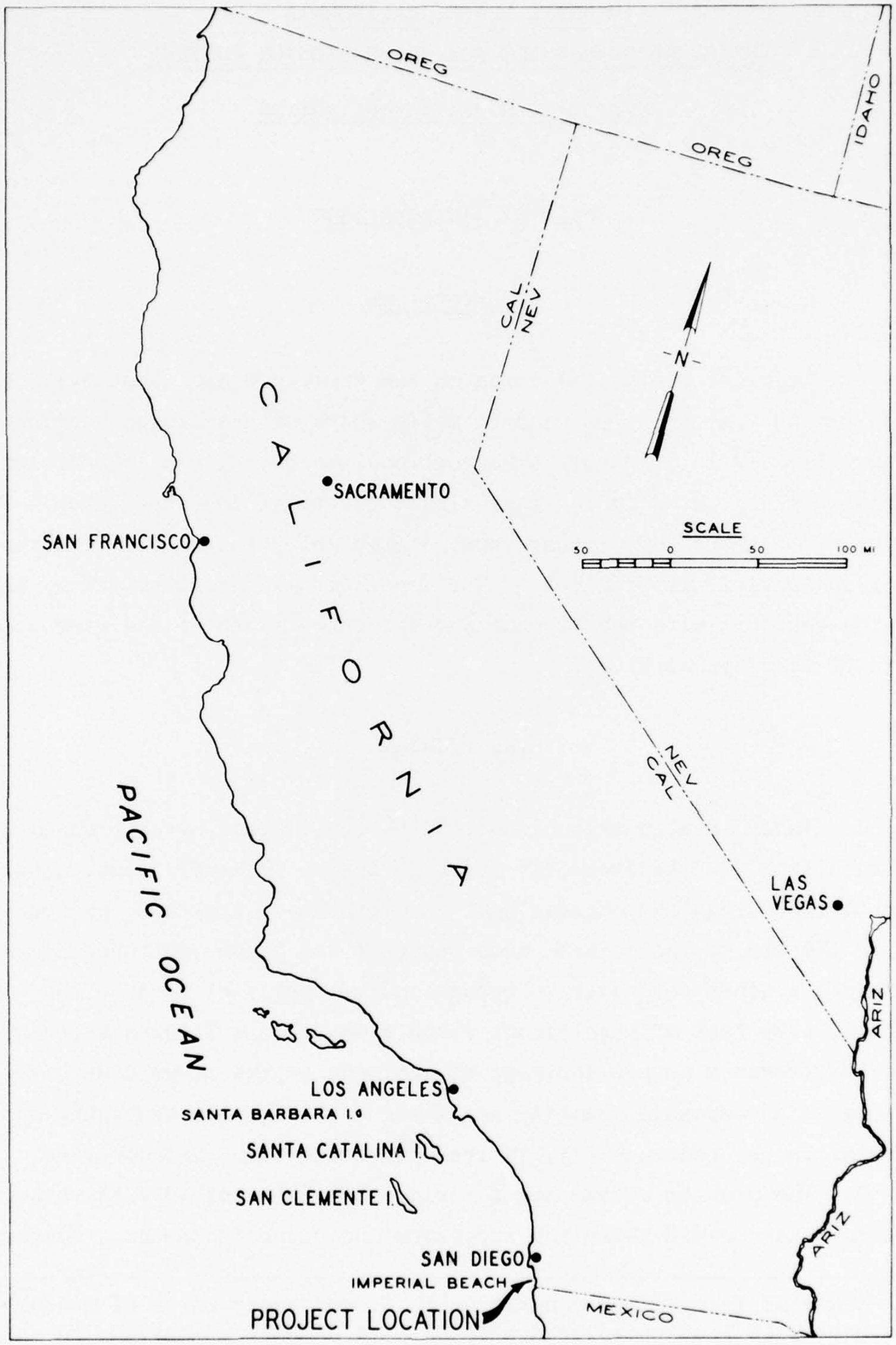

Figure 1. Vicinity map

storms during the next several years forced local and private interests to install a stone revetment along the shoreline. Between the years 1959 and 1963, the Corps of Engineers constructed two groins in the area. However, these have been ineffective in restoring the beach, and stone revetments have been installed in several locations by private interests.

4. Approximately one million cubic yards of beach material was dredged from San Diego Harbor and disposed of on the Imperial Beach shoreline from March through June 1977. Improvements are desperately needed to provide adequate protection for this beach fill.

Purposes of the Model Study

5. The purposes of the model study were to:

   a. Determine the mechanisms by which sand is being lost from the existing beach.

   b. Study shoaling and wave conditions with the proposed improvement plan installed in the model.

   c. Develop alternative remedial plans for the alleviation of undesirable conditions as found necessary.

   d. Determine whether suitable design modifications of the proposed plans could be made that would reduce construction costs significantly and still provide adequate protection.

PART II: THE MODEL

Design of the Model

6. The Imperial Beach model was constructed to a linear scale of 1:75, model to prototype. Scale selection was based on the following factors:

- a. Depth of water required in the model to prevent excessive bottom-friction effects.
- b. Absolute size of model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Capabilities of available wave-generating and wave-measuring equipment.
- f. Cost of model construction.

A geometrically undistorted model was necessary to ensure accurate reproduction of short-period wave patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law.[2] Scale relations used for design and operation of the model were as follow:

Characteristics	Dimensions*	Model:Prototype Scale Relations
Length	L	$L_r = 1:75$
Area	L^2	$A_r = L_r^2 = 1:5625$
Volume	L^3	$\forall_r = L_r^3 = 1:421,875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$

* Dimensions are in terms of length and time.

7. Ideally, a quantitative, three-dimensional, movable-bed model investigation would best determine the effectiveness of various project plans for the prevention of beach erosion at Imperial Beach. However,

this type of model investigation is difficult and expensive to conduct, and each area in which such an investigation is contemplated must be carefully analyzed. The following computations and prototype data are considered essential for such investigations[3]:

a. A computation of the littoral transport, based on the best available wave statistics.

b. An analysis of the sand-size distribution over the entire project area (offshore to a point well beyond the breaker zone).

c. Simultaneous measurements of the following items over a period of erosion and accretion of the shoreline (this measurement period should be judiciously chosen to obtain the maximum probability of both erosion and accretion during as short a time span as possible):

(1) Continuous measurements of the incident wave characteristics. Such measurements would mean placing enough redundant sensors to accurately estimate the directional spectrum over the entire project area, and in addition, would mean conducting a rather sophisticated analysis of all these data.

(2) Bottom profiling over the entire project area using the shortest time intervals possible.

(3) Nearly continuous measurements of both littoral and onshore-offshore transport of sand. These measurements would be especially important over the erosion-accretion period. A wave forecast service would be essential to this effort to prepare for full operation during the erosion period.

8. In view of the complexities involved in conducting movable-bed model studies and due to limited funds and time for the Imperial Beach project, the model was molded in cement mortar (fixed-bed) at an undistorted scale of 1:75 and a tracer material was obtained to determine qualitatively the degree of sediment movement for various plans.

9. Model limits were selected to allow reproduction of as much shoreline on each side of the study area as possible while keeping construction costs at a reasonable level. The main considerations were that the updrift shoreline be long enough to allow proper formation of the longshore currents before reaching the study area and that the downdrift shoreline be of sufficient length to prevent deflection of the longshore currents back into the study area.

10. For development of a longshore current along a straight beach, Eagleson[4] gave the following relation:

$$u_L^2(x) = A - [A - u_L^2(0)]e^{-Bx}$$

where $u_L(x)$ = the longshore current at distance x

$u_L(0)$ = the longshore current at the origin $x = 0$

$$A = \frac{3}{8} \frac{gH_b^2 n_b}{d_b} \sin\alpha \sin\theta_b \sin 2\theta_b / f$$

$$B = \frac{2}{5} [f/(d_b \cos\alpha \sin\theta_b)]$$

H_b = the wave height at breaking

$$n_b = \frac{1}{2} [1 + 2k_b d_b / \sinh 2k_b d_b]$$

d_b = the water depth at breaking

α = the beach slope

θ_b = the angle of incident wave at breaking

f = the Darcy-Weisbach friction factor suggested to be:

$$f = [2 \log_{10}(\frac{d_b}{k_*}) + 1.74]^{-2}$$

where k_* is the absolute roughness of the beach surface

The longshore current velocity u_L tends to a constant velocity $A^{1/2}$ as the value of Bx grows. For $Bx = 2$, $u_L = 0.92A^{1/2}$.

11. Since the contours at Imperial Beach are relatively straight in the breaker zone, Eagleson's equation was applied for a range of incident waves to determine approximate distances for formation of longshore currents. For a typical example (beach slope = 1°10', breaker angle = 5°, wave height at breaking = 7 ft, and depth at breaking = 9 ft), 345 ft of updrift shoreline would be required. Similar computations for the full range of incident waves produced distances of the same order of magnitude. The shoreline distances of 2700 ft and 4200 ft reproduced to the north and south of the study area, respectively, and the wave

generator length extending 1000 ft to the north and to the south of the study area, therefore, were considered more than adequate for proper formation of longshore currents.

12. Preliminary model tests were conducted to determine the effects of model boundaries on longshore currents. It was found that the longshore current was deflected seaward into deeper water at the downdrift boundary. It then moved behind the wave generator to the updrift boundary forming a somewhat natural recirculation system. No model boundary effects on the longshore current were noted in the study area for at least 15 min (model time). The longshore currents reached equilibrium very rapidly (within about 30 sec), and all model data were taken within the first 8 min as a precaution against possible model boundary effects.

13. Based on the principles of hydraulic similitude, the model correctly reproduced:

- a. Wave refraction.
- b. Wave shoaling.
- c. Wave diffraction.
- d. Wave breaking.
- e. Nearshore circulation cells (rip, feeder, and eddy currents).
- f. Longshore currents generated by breaking waves (within the area covered by the wave generator).
- g. Qualitative sediment transport in the breaker zone.

14. The model did not reproduce longshore currents and sediment transport at the model boundaries. Some of the problems associated with reproducing longshore currents and/or sediment transport at the model boundaries were:

- a. Lack of prototype data.
- b. Complexity of nearshore circulation cells and longshore current patterns.
  - (1) Longshore currents are interrupted by rip currents and eddies.
  - (2) Wave refraction causes areas of energy convergence and divergence.

(3) Wave refraction changes the breaking wave angle.

c. Operational problems.

(1) Lack of information about the proper location and design of circulation systems at model boundaries, current distribution, and friction effects

(2) Boundary conditions change for each test wave and direction (this would require in addition that extensive prototype data be acquired to properly attempt to reproduce this boundary condition).

15. The proposed improvement plans for Imperial Beach included the use of rubble-mound breakwaters and groins. Experience and experimental research have shown that considerable wave energy passes through the interstices of this type of structure; thus, the transmission and absorption of wave energy became a matter of concern during design of the 1:75-scale model. In small-scale models, rubble-mound structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures.[5] Also, the transmission of wave energy through the structure is relatively less for the small-scale model than for the prototype. Consequently, some adjustment in small-scale rubble-mound structures is needed to ensure satisfactory reproduction of wave-reflection and wave-transmission characteristics. In past investigations[6,7] at the U. S. Army Engineer Waterways Experiment Station (WES), this adjustment was made by determining the wave-energy transmission characteristics of the proposed structure in a two-dimensional model using a scale large enough to ensure negligible scale effects. A breakwater section then was developed for the small-scale three-dimensional model that would provide essentially the same relative transmission of wave energy. Therefore, from previous findings for breakwaters and wave conditions similar to those at Imperial Beach, it was determined that a close approximation of the correct wave-energy transmission characteristics could be obtained by increasing the size of the rock used on the 1:75-scale model to approximately one and a half that required for geometric similarity. Accordingly, in constructing the structures in the Imperial Beach model, the rock sizes were computed linearly by scale, then multiplied by 1.5 to determine the actual sizes to be used in the model.

The Model and Appurtenances

16. The model (Figure 2) reproduced 2.6 miles of shoreline and underwater contours to offshore depths ranging from -36 to -45 ft, with a sloping transition to the wave-generator pit elevation of -70 ft. The total model area of 25,500 sq ft represented about 5.2 square miles in the prototype. The model with existing conditions installed is shown in Photo 1. Vertical control for model construction was based on the mean lower low water (mllw) elevation* of 0.0 ft. Horizontal control was referenced to a local prototype grid system.

17. Model waves were generated by a 115-ft-long wave generator with a flat-faced horizontal-motion plunger. The horizontal movement of the plunger caused a periodic displacement of water incident to this motion. The length of stroke and the period of the horizontal motion were variable over the range necessary to generate waves with the required characteristics. In addition, the wave generator was mounted on retractable casters which enabled it to be positioned to generate waves from the required directions.

18. An Automatic Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 3) was used to secure wave-height data at selected locations in the model. Basically, through the use of a minicomputer, ADACS recorded onto magnetic tape the electrical output of parallel-wire, resistance-type sensors. These sensors measured the change in water-surface elevation with respect to time. The magnetic tape output of ADACS was then analyzed to obtain the wave-height data.

19. A 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to damp any wave energy that might otherwise be reflected from the model walls. In addition, guide vanes were placed along the wave generator sides to ensure proper formation of the wave train incident to the model contours.

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* All elevations (el) cited herein are in feet referred to mean lower low water.

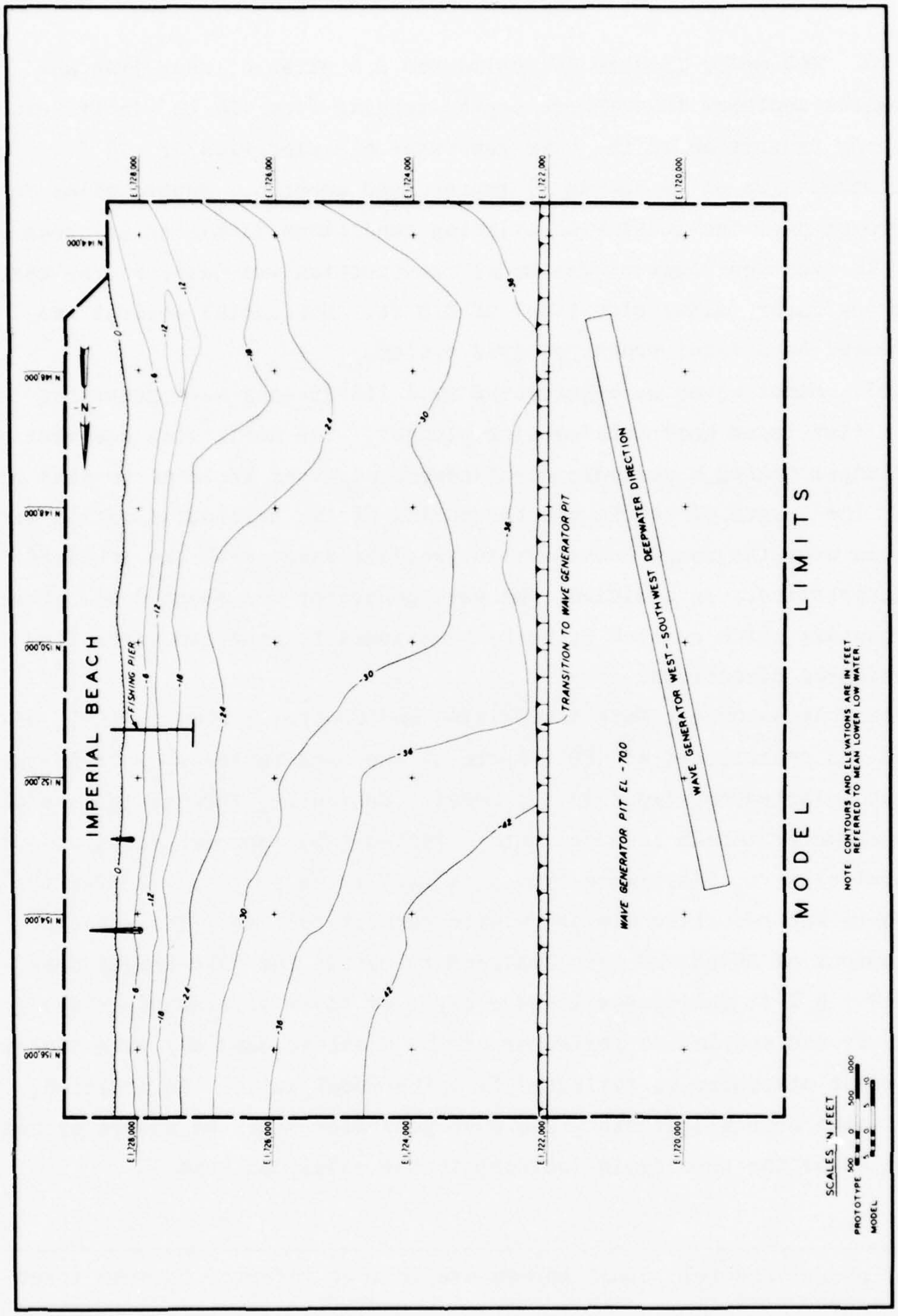

Figure 2. Model layout

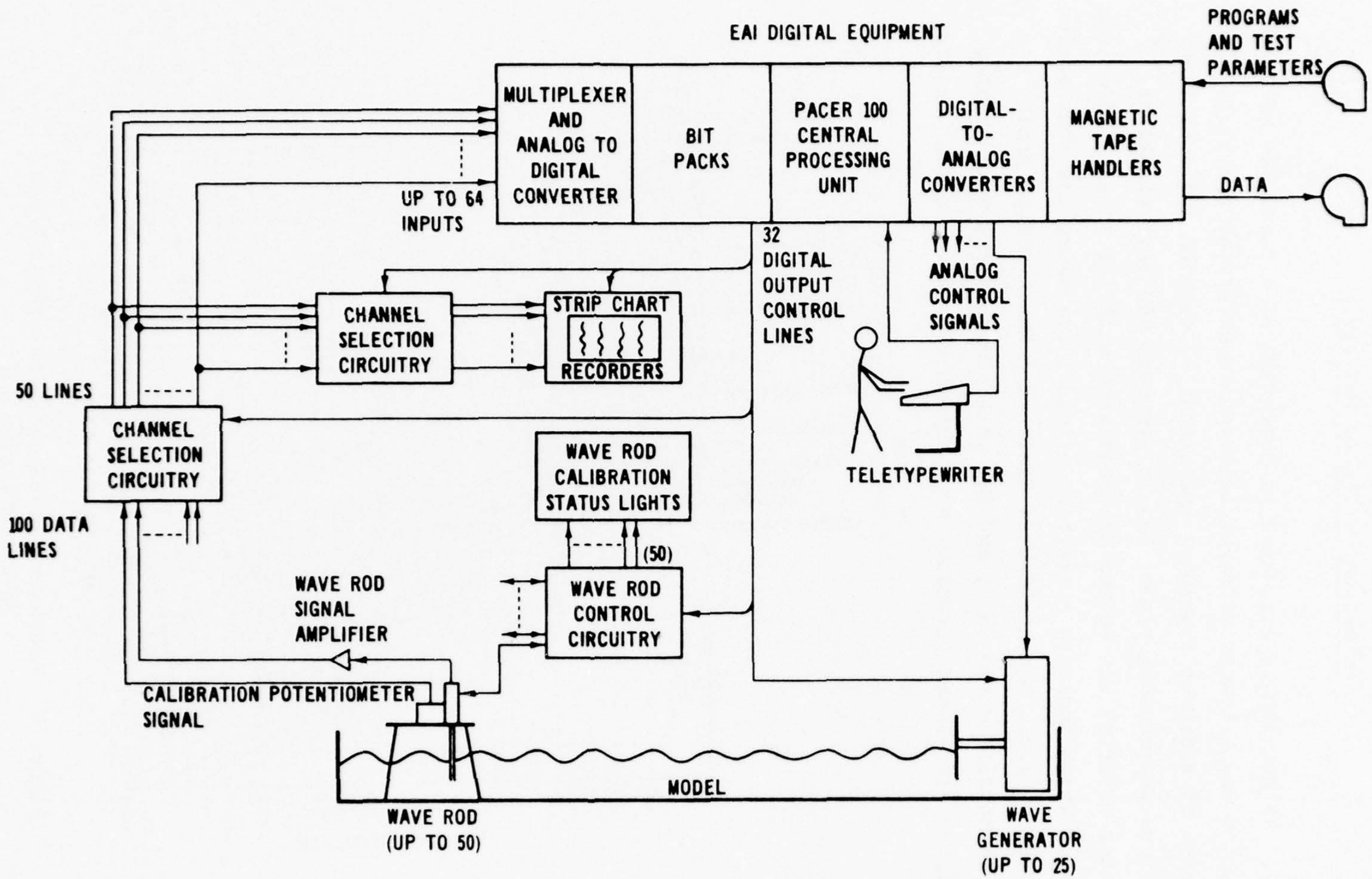

Figure 3. Automated Data Acquisition and Control System (ADACS)

Selection of Tracer Material

20. As previously discussed in paragraph 8, a fixed-bed model was constructed and a tracer material selected to determine qualitatively the degree of sediment transport and extent of erosion and accretion for various improvement plans. As in previous WES investigations,[8,9] the tracer material was chosen in accordance with the scaling relations of Noda,[10] which indicate a relation or model law among the four basic scale ratios, i.e., the horizontal scale λ ; the vertical scale μ ; the sediment size ratio η_D ; and the relative specific weight ratio η_γ' (Figure 4). These relations were determined experimentally using

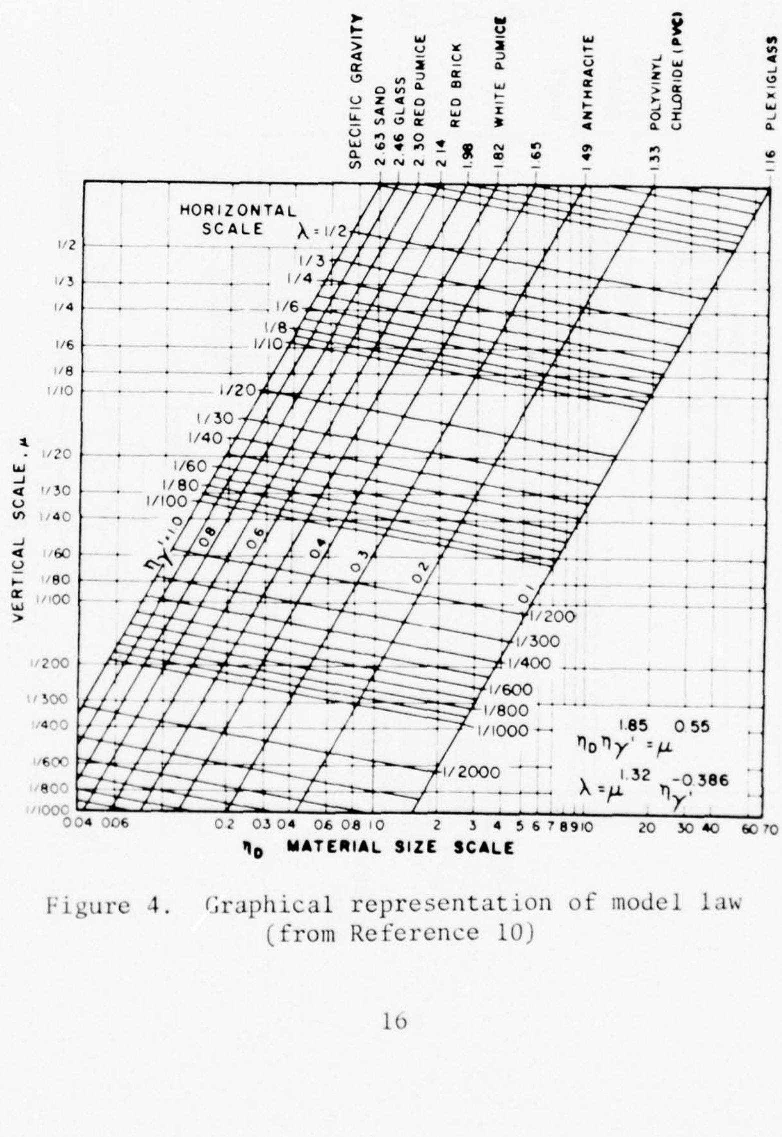

Figure 4. Graphical representation of model law (from Reference 10)

a wide range of wave conditions and beach materials and are valid mainly for the breaker zone.

21. Noda's scaling relations indicate that movable-bed models with scales in the vicinity of 1:75 (model to prototype) should be distorted (i.e., they should have different horizontal and vertical scales). Since the fixed-bed model of Imperial Beach was undistorted to allow accurate reproduction of sea and swell and wave-induced currents, the following procedure was used to select a tracer material. Using the prototype sand characteristics (median diameter, D_{50} = 0.20 mm; specific gravity = 2.65) and assuming the horizontal scale to be in similitude (i.e., 1:75), the median diameter for a specific gravity of a given tracer material and the vertical scale were computed. The vertical scale was then assumed to be in similitude, and the tracer median diameter and horizontal scale were computed. This resulted in a range of tracer material sizes for given specific gravities that could be used. A search was made of all movable-bed materials at WES, preliminary model tests were conducted, and a quantity of crushed coal (specific gravity = 1.30, median diameter, D_{50} = 0.55 mm) was selected for the tracer tests.

PART III. TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

22. Still-water levels (swl) for wave-action models are selected so that various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include refraction of waves as they approach the beach, overtopping of structures by waves, reflection of wave energy from structures, and transmission of wave energy through porous structures.

23. From U. S. Coast and Geodetic Survey records (now, National Ocean Survey),[11] the mllw level at Imperial Beach is 0.0 ft, and the mean higher high water (mhhw) level is +5.4 ft. The mhhw stage was considered to be representative of water levels to be expected during a severe storm and a swl of +5.4 ft was selected for use in the model. The mllw level of 0.0 ft was also selected for use in the model to ensure that the relative effectiveness of the various plans was not too sensitive to the swl.

Wave Dimensions and Directions

Factors influencing selection of test-wave characteristics

24. In planning the test program for a model investigation of wave-action problems, it is necessary to select dimensions and directions for the test waves that will afford a realistic test for the proposed improvement plans and allow an accurate evaluation of the elements of the various proposals. Surface wind waves are generated by the tangential shear force of the wind blowing along the water surface and the normal force of the wind blowing against the wave crests. The magnitude of the maximum wave that can be generated by a given storm depends on the wind speed, the length of time that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows. Selection of test wave conditions entails evaluation of such factors as:

a. Fetch and decay distances (the latter being the distance over which waves travel after leaving the generating area) for the various directions from which waves can attack the problem area.

b. Frequency of occurrence and duration of storm winds from the different directions.

c. Alignment and relative geographic position of the study area.

d. Alignments, lengths, and locations of various structures in the study area.

e. Refraction of waves caused by differentials in depths in the area seaward of the study area, which may cause either a convergence or a divergence of wave energy.

Wave refraction

25. When wind waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to selection of test-wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. Changes in wave height and direction can be determined by plotting refraction diagrams and calculating refraction coefficients. These diagrams are constructed by plotting the position of wave orthogonals (lines drawn perpendicular to wave crests) from deep water into shallow water. If it is assumed that the waves do not break and that there is no lateral flow (diffraction) of energy along the wave crest, the ratio between the wave height in deep water (H_o) and the wave height in shallow water (H) will be inversely proportional to the square root of the ratio of the corresponding orthogonal spacings (b_o and b) or $H/H_o = K(b_o/b)^{1/2}$. The quantity $(b_o/b)^{1/2}$ is the refraction coefficient; K is the shoaling coefficient. Thus, the refraction coefficient multiplied by the shoaling coefficient gives a conversion factor for transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, which is a function of wavelength and water depths, can be obtained from Reference 12.

26. Wave-refraction diagrams were furnished by the U. S. Army Engineer District, Los Angeles (SPL), for deepwater wave directions

ranging from 240° to 310° and wave periods ranging from 4 to 20 sec. These diagrams represented the propagation of the wave fronts from deep water to shallow water (to the point of breaking). By positioning the wave generator to correspond with the wave front at -70 ft (the elevation of the wave-generator pit), the refracted wave from the deepwater direction was accurately reproduced.

Prototype wave data and selection of test waves

27. Deepwater wave hindcast data by Marine Advisors[13] for Stations A and C were selected for use in the model. Station A was used to represent the west and west-southwest directions. Due to the positioning of San Clemente and Santa Catalina Islands, Station C was used for the west-northwest direction. These data (Table 1) represent the estimated durations and magnitudes of deepwater waves (sea and swell) approaching Imperial Beach from various directions. The refraction-shoaling analysis described in paragraph 26 was used to transfer the deepwater waves into shallow water for use in the model (Table 2). The shallow-water wave directions used in the model were the average directions of the refracted waves for the significant wave periods noted from each deepwater wave direction. The characteristics of the test waves used in the model were selected from Table 2, as shown in the following tabulation:

Deepwater Wave Direction	Selected Shallow-Water Wave Test Direction	Selected Test Wave Period sec	Selected Test Wave Height ft
	swl = +5.4 = mhhw		
S60°W (240°)	256°	7	4, 11
		10	4, 7
		14	4, 7
		18	4, 11
West (270°)	270°	7	4, 11
		10	4, 15
		14	4, 9
		18	4, 15
N60°W (300°)	281°	7	4, 7
		10	4, 9

(Continued)

Deepwater Wave Direction	Selected Shallow-Water Wave Test Direction	Selected Test Wave Period sec	Selected Test Wave Height ft
	swl = 0.0 = mllw		
S60°W (240°)	256°	7	4, 11
		14	4, 7
West (270°)	270°	7	4, 11
		14	4, 9
N60°W (300°)	281°	7	4, 7
		10	4, 9

28. Selection of test waves took place at the beginning of this study, and the Marine Advisors data were the best available statistics at the time. Recently, hindcasts based on 20 years of data[14] have been completed and probably are more representative of the actual wave climate since the Marine Advisors data were based on 3-year hindcasts. Both data sets have shortcomings, specifically regarding the southern hemisphere swell, but they are the best available at this time.

Analysis of Model Data

29. The relative merits of various plans tested were evaluated using (a) comparison of wave heights at selected locations in the study area, (b) comparison of current patterns and magnitudes, (c) movements of tracer material, and (d) visual observations and photographs. In the wave-height data analysis, the average of the highest one-third of the waves (significant wave height) at each gage location was selected. By using Keulegan's equation,[15] the reduction of wave heights in the model due to bottom friction was calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel. For nonbreaking waves, Keulegan's factor (K_f) was multiplied times the measured value shown to obtain an adjusted wave height at a particular gage. This procedure could not be followed for breaking waves (depth limited), however, because the resulting wave height would be larger than that possible for the water depth at that gage. In such

cases, the actual measured value was used and the incident wave (at the wave generator) adjusted by dividing by K_f .

30. Since the purpose of this study was to develop a plan to prevent the loss of beach material from the study area, movement of tracer material was a prime concern. These tests are of a qualitative nature, and no estimates of quantities transported are possible. Any appreciable loss of tracer material, therefore, was considered a problem; and only those plans with very little or no loss of tracer material were considered as viable alternatives.

PART IV: TESTS AND RESULTS

Description of Tests

Existing conditions

31. Prior to tests of various improvement plans, comprehensive tests were performed for existing conditions (Plate 1). Wave-height data were obtained for various stations along the proposed breakwater center line for the test conditions listed in paragraph 27. Wave-induced current patterns and magnitudes and tracer patterns also were secured for representative waves from the three selected test directions.

32. Current patterns and magnitudes and tracer patterns also were secured for existing conditions with groins 1 and 2 removed (Plate 2) to determine whether these groins had any adverse effects on the littoral processes.

Improvement plans

33. Current pattern and magnitude tests and tracer tests were conducted for 19 improvement plan variations. These variations consisted of changes in the lengths and alignments of the breakwater structures, changes in the breakwater cross section, and changes in breakwater and groin locations. Photographs of wave patterns with current patterns and magnitudes superimposed and photographs of tracer patterns were obtained for all major improvement plans. Brief descriptions of the improvement plans are presented in the following subparagraphs; dimensional details are presented in Plates 3-10.

- a. Plan 1 (Plate 3) consisted of a segmented rubble-mound breakwater with five 700-ft-long sections spaced 700 ft apart and a crown elevation of -2.0 ft. These were constructed in a straight line at approximately the -15.0 ft contour and extended southward from a point 350 ft south of groin 1.
- b. Plan 1A (Plate 3) entailed the elements of Plan 1 with the gaps between the individual breakwater sections filled to make a continuous breakwater with a -2.0 ft crown elevation.
- c. Plan 1B (Plate 3) was the same as Plan 1 but with the crown elevation raised to +5.4 ft.

d. Plan 1C (Plate 3) entailed the elements of Plan 1B with a 100-ft extension at each end of the breakwater segments (900-ft-long segments).

e. Plan 1D (Plate 3) consisted of the elements of Plan 1C with a 50-ft extension at each end of the breakwater segments (1000-ft-long segments).

f. Plan 1E (Plate 3) involved the elements of Plan 1D with each breakwater segment shifted 150 ft to the north.

g. Plan 2 (Plate 4) consisted of a segmented rubble-mound breakwater with nine 350-ft-long sections spaced 350 ft apart and a crown elevation of +5.4 ft. These were constructed in a straight line at the -15.0 ft contour and extended southward from a point 350 ft south of groin 1.

h. Plan 3 (Plate 5) consisted of a 6650-ft-long continuous breakwater with a crown elevation of 0.0 ft. It was located at about the -10.0 ft contour with groin 1 extended to tie into the north end of the structure.

i. Plan 3A (Plate 5) involved the elements of Plan 3 with a groin having a crown elevation of 0.0 ft tied into the breakwater 350 ft north of the southern end.

j. Plan 4 (Plate 6) consisted of a 7000-ft-long stepped breakwater situated at about the -10.0 ft contour. The structure alternated between five 700-ft long breakwaters and five 700-ft-long sills with crown elevations of 0.0 ft and -5.0 ft, respectively. The structure was constructed with a breakwater centered at the fishing pier so that 3150 ft lay north and 3850 ft lay south of the pier. Groin 1 was extended to tie into the structure. At the south end of the structure, a groin was constructed to tie into the center of the sill. Its crown elevation sloped from +5.0 ft at the shoreline to -5.0 ft at the structure.

k. Plan 4A (Plate 6) entailed the elements of Plan 4 with the sills made impervious.

l. Plan 4B (Plate 6) involved the elements of Plan 4A with groin 2 removed.

m. Plan 4C (Plate 6) consisted of the elements of Plan 4A with groins 1 and 2 removed.

n. Plan 5 (Plate 7) consisted of a segmented breakwater made of ten 350-ft-long sections with a crown elevation of +5.0 ft spaced 300 ft apart and situated along the -5.0 ft contour. A segment was placed at the end of each groin with another segment centered between them.

o. Plan 5A (Plate 7) involved the elements of Plan 5 with a 0.0-ft crown elevation sill placed in the breakwater gaps.

p. Plan 5B (Plate 7) involved the elements of Plan 5 with the crown elevation raised to +10.0 ft.

q. Authorized Groin Plan (Plate 8) involved the elements of existing conditions with three additional groins placed 800 ft north, 200 ft south, and 1430 ft south of the pier and extending to about the -6.0 ft contour. The groins were constructed with a +12.0 ft crown elevation to within 150 ft of the groin head where the elevation sloped down to +7.0 ft.

r. Plan 6 (Plate 9) consisted of nine sheetpile groins which were 2 ft high and extended to the -12.0 ft contour.

s. Plan 7 (Plate 10) consisted of nine rubble-mound groins of the same length and position as in Plan 6. Crown elevations were +12.0 ft to within 150 ft of the groin head then sloped down to +7.0 ft.

The proposed beach fill at Imperial Beach was not added for the above plans. Since the installation of this fill will substantially precede the construction of protective structures (1 to 3 years), it was assumed that the wave environment would rearrange the fill so that underwater beach slopes would be similar to those now existing at Imperial Beach. The only difference would be a widening of the beach berm. Consequently, the proposed structures were constructed in the model on the existing contours with locations referenced to a particular contour (i.e., the -15 ft, the -10 ft, or the -5 ft contour).

Wave-height tests

34. Wave-height tests for existing conditions and various improvement plans were conducted using test waves from one or more of the test directions listed in paragraph 27. As an expedient, tests involving certain proposed improvement plans were limited to one critical direction of wave approach (i.e., the S60°W deepwater direction). From previous tests, it became apparent that the S60°W direction was one of the most critical directions of wave approach for wave heights, current magnitudes, and depletion of tracer material from the protected area. Existing conditions and Plan 4 were tested comprehensively from all directions. Following Plan 5B, wave-height tests were discontinued as an expedient for the development of additional improvement plans. It became apparent that the most critical and sensitive tests performed

for plan evaluation were the tracer tests. If no material (or a relatively negligible amount) left the protected area, then the plan was considered good. Furthermore, the current measurements in conjunction with the tracer tests were generally more indicative of a plan's relative value. One reason for this is the difficulty in measuring wave heights in areas of wave breaking and nonlinear transformations. A gage may be placed precisely at the peak breaking point one time and slightly before or after this location another time, making the comparison of wave heights not necessarily the most solid measure of the plan's effectiveness. The wave gage locations for existing conditions and each improvement plan are shown in Plates 1 and 3-7.

Current pattern and magnitude tests

35. Wave-induced current patterns and magnitudes were determined at selected locations by timing the progress of a dye tracer relative to a known distance on the model surface. These tests were conducted for existing conditions and the various improvement plans using the same test directions and test waves as for the wave-height tests.

Tracer tests

36. Tracer tests were conducted for existing conditions and the various improvement plans using the same test directions and test waves as for the wave-height tests. Before each test, tracer material was placed in rows extending from the shoreline to either the breakwater or through the surf zone. These rows were arranged on a prototype grid system spaced 500 ft apart.

37. Semimovable-bed tracer tests were performed for existing conditions, Plan 5A, and all variations of Plan 4. The study area was blanketed by a uniform layer of crushed coal (approximately 1/2 in. thick) to determine the effect of the structures on the shoreline and their ability to contain the fill material. Semimovable-bed tracer tests were conducted for existing conditions to establish a base with which to compare the relative effectiveness of the various plans. These are described as semimovable-bed tests in that they reproduce areas of accretion and erosion, but the extent of the latter is limited to 1/2 in. (approximately 3 ft prototype) by the concrete floor. Using waves from

the S60°W deepwater direction, the testing order of the waves and their total duration were as follows:

Period sec	Height ft	Total Duration Model min	Total Duration Prototype hr
7	4	30	4.3
7	11	20	2.9
14	4	30	4.3
14	7	20	2.9

To prevent any scale effects from model circulation due to the artificial model boundaries, each continuous test run was limited to 15 min for the small waves and 10 min for the larger waves.

Motion picture footage

38. Motion picture footage of the Imperial Beach model with existing conditions, the Authorized Groin Plan, Plan 4, and Plan 5A installed in the model was secured and forwarded to SPL for use in a film about the Imperial Beach Project, using 7-sec, 11-ft waves from the S60°W deepwater direction. Included in the movie footage were the following:

a. For existing conditions, a general view and closeups of current patterns and closeups of semimovable-bed tracer tests.

b. For the Authorized Groin Plan, a general view and closeups of current patterns.

c. For Plan 4, a general view and closeups of current patterns and semimovable-bed tracer tests.

d. For Plan 5A, a general view and closeups of current patterns and closeups of semimovable-bed tracer tests.

Test Results

39. In evaluating test results, the relative merits of each plan were based primarily on an analysis of wave heights, the movement of tracer material and subsequent deposits, and current patterns and magnitudes. From this evaluation, the best improvement plans were selected.

Existing conditions

40. Wave heights for existing conditions are presented in

Tables 3 and 4 for swl's of +5.4 and 0.0 ft, respectively. The maximum wave height recorded for mhhw was 18.0 ft (gage 11) and for mllw was 13.8 ft (gage 12).

41. Current patterns and magnitudes secured for existing conditions (Photos 2-9) reveal that wave-induced currents for the west and N60°W test directions generally flowed from north to south. For the S60°W test direction the currents generally flowed from south to north. For all directions, the currents in the vicinity of the groins were generally confused with numerous rip currents and eddies. Some of these rip currents reached magnitudes as high as 5.8 fps. To the south of the fishing pier, current patterns were less confused. Longshore currents, interrupted at intervals by rip currents, ran as high as 5.0 fps. In some cases, the location and magnitude of rip currents were fairly constant; in some cases, they were transient. In general (regardless of wave characteristics), there was usually one strong rip current in the vicinity of groins 1 and 2, one in the vicinity of the fishing pier, and one to the south of the fishing pier. The permanent rips are probably bathymetrically controlled (i.e., the rip to the south of the pier is probably controlled by the presence of the Tijuana Shoal). The presence of the two groins also contributes to the formation of rips. The transient rips are probably controlled to a major extent by changing wave conditions.

42. Tracer tests showed a general shoreward movement of material for small, low-steepness waves and a seaward movement for large, high-steepness waves. The eddies tended to trap tracer material while the rip currents tended to carry tracer material seaward. For the west and N60°W test directions, longshore currents moved tracer material to the south, while for the S60°W test direction, tracer material moved to the north. When the tracer material encountered rip currents, especially those for the larger test waves, significant amounts of material moved seaward into relatively deep water where it probably would not be recovered by the natural littoral processes. Tracer patterns are presented in Photos 10-18.

43. Semimovable-bed tracer tests (testing order shown in

paragraph 37) for existing conditions (Photos 19-23) were conducted to establish a base with which to compare the relative effectiveness of the various plans. For large waves from S60°W, it was observed that the tracer material formed an offshore bar which rapidly migrated to the north. However, material shoreward of the breaker zone continued to accrete the beach. The amount of coal (tracer material) lost from the study area was approximately 14 percent. To determine the effect of the order of the waves on test results, the coal was reshaped; and a 7-sec, 11-ft wave was tested first. The result was a coal loss of 18 percent and no beach accretion. This test indicates that the onshore-offshore movement of tracer material is sensitive to wave size and order of testing.

44. Current patterns and magnitudes for existing conditions with the groins removed showed the continued formation of rip currents in this area for each wave tested (Photo 24). This would indicate that these rips are bathymetrically controlled; however, the groins do contribute to the exact location and the magnitude.

45. Tracer patterns for existing conditions (Photo 25) with the groins removed showed continued loss of tracer material in this area, especially for the large waves.

Improvement plans

46. Using wave data obtained for existing conditions (Tables 3 and 4), design procedures from the Shore Protection Manual (SPM),[12] and test results from the Perched Beach Study (1971),[3] an initial cross-sectional design of the submerged breakwaters was obtained. The SPM design for a submerged rubble foundation resulted in an armor stone size of about 6.5-8.5 tons (depending on water level), and an extrapolation of data from the Perched Beach Study resulted in an armor stone size of about 7-9 tons. An armor stone size of 8 tons was selected as a reasonable first approximation, and the resulting breakwater cross section is shown in Plate 11. This stone size was adjusted to compensate for scale effects according to the procedure outlined in paragraph 15, and was used in both breakwater structures and low-sill structures for Plans 1-5B.

47. Wave heights for Plan 1 (Table 5) for the S60°W deepwater

direction, reached a maximum height of 17.1 ft at gage 7 (Plate 3) for mhhw and 7.7 ft at gage 7 for mllw.

48. Current patterns and magnitudes for Plan 1 are shown in Photos 26-28, and velocities were up to 4.3 fps with rip currents exiting through the breakwater gaps and around groin 1.

49. Tracer patterns showed continued longshore transport shoreward of the structures, loss of tracer material through the gaps of the breakwater in the vicinity of the groins, and loss of material around the end of groin 1 (Photos 29-32).

50. As an expedient to develop a more promising plan, Plans 1A-1E were tested using tracer material and dye for the 7-sec, 11-ft wave from the S60°W deepwater direction only (most critical for loss of tracer material from the study area). As the crown elevation was raised and the breakwater length increased (gaps reduced), the tracer material began to accrete. However, the volume of rock required for construction (Table 12) of these structures increased drastically (consequently increasing costs significantly).

51. Wave-height measurements for Plan 2 (Table 6) for the S60°W deepwater direction revealed maximum wave heights of 17.0 ft at gage 7 (Plate 4) for mhhw and 8.9 ft at gage 3 for mllw.

52. Current pattern photographs for Plan 2 (Photos 33 and 34) reveal general longshore currents to the north interrupted at intervals by rip currents. The rip currents exited through the breakwater gaps as in Plan 1 but were more numerous and weaker in magnitude than those for Plan 1.

53. Tracer tests (Photos 35-37) showed material loss only for the 7-sec, 11-ft waves at mhhw. Tracer material left the study area in a rip current south of the fishing pier and also around groin 1.

54. Initial tests of Plan 3 indicated that for the 7-sec, 11-ft waves from the S60°W deepwater direction, a strong southerly longshore current formed south of the fishing pier, transporting significant quantities of tracer material out of the south end of the study area. Therefore, it was necessary to install a groin at the south end of the breakwater (Plan 3A).

55. Wave-height measurements for Plan 3A for the S60°W deepwater direction are presented in Table 7. The maximum wave height for mhhw was 12.9 ft at gage 1 (Plate 5); for mllw, wave heights were 6.1 ft at gages 1 and 2. This plan, when compared with existing conditions, effects average reductions in wave heights of about 13 percent for the mhhw tests and about 27 percent for the mllw tests.

56. Results of the current tests for Plan 3A (Photos 38-40) showed reduced velocities for both longshore currents and rip currents. It was observed that the new groin at the south end of the breakwater effectively stopped the strong longshore currents previously observed in that area for Plan 3.

57. Tracer tests for Plan 3A (Photos 41-43) showed a general onshore movement of material shoreward of the structure for all test waves. Some longshore transport developed between groin 2 and the fishing pier for the larger waves; however, no material was lost from the enclosed area.

58. Wave-height measurements for Plan 4 (Tables 8 and 9) showed maximum wave heights of 15.7 ft at gage 2 (Plate 6) for the 18-sec, 15-ft wave at mhhw and 10.3 ft at gage 2 for the 14-sec, 9-ft wave at mllw. Both waves were from the west deepwater direction. A comparison of average wave heights for existing conditions and Plan 4 showed average reductions of 14 and 24 percent for mhhw and mllw tests, respectively.

59. Current patterns and magnitudes for Plan 4 for the N60°W and west deepwater directions (Photos 44-49) showed a general longshore current to the south interrupted periodically by rip currents and eddies (particularly in the northern reach of the study area). The rip currents were generally weaker than those for existing conditions. Test results for waves from the S60°W deepwater direction (Photos 50-52) were similar but with a general northerly longshore current. In all cases, waves broke over the higher breakwater sections (crown el 0.0 ft). Currents then flowed along the shore to the lower breakwater sections (crown el -5.0 ft) where seaward flowing rip currents were formed. The magnitudes of these rip currents were higher than for Plan 3A but less than those for existing conditions.

60. Results of tracer tests for Plan 4 (Photos 53-55) for the N60°W deepwater direction showed a general onshore and sometimes southerly movement of material. With the exception of the 10-sec, 9-ft wave at mhhw, the rip currents were generally too weak to move an appreciable quantity of material. For the west (Photos 56-58) and S60°W (Photos 59-61) deepwater direction, tracer movement was also generally onshore. Rip currents for the large waves at mhhw carried some material toward the structure but not over it. In all cases, no material was lost from the study area with the exception of a small amount, initially placed adjacent to the structure, which was transmitted seaward through the voids of this structure; however, this was not considered to be a significant problem.

61. Semimovable-bed tracer tests were performed for Plan 4 for the S60°W deepwater direction using test waves described in paragraph 37. For the 7-sec, 4-ft waves (Photo 63) the formation of bars in the lee of the high structures was observed. A comparison with Photo 62 shows the increase in beach berm width. There was essentially no loss of material for this wave. For the 7-sec, 11-ft wave (Photo 64) the bars continued to grow while some welded to the beach. Some loss of coal (tracer material) from the enclosed area was observed (primarily north of the pier). The coal migrated to the sill structure where some passed through and over the structure until an equilibrium condition was established. Subsequently, with each wave, coal oscillated back and forth over the sill. For the 14-sec, 4-ft wave, accretion accelerated. The bars welded to the beach, and some coal returned to the study area from outside the structures. The 14-sec, 7-ft wave proved to be the worst condition for Plan 4 (Photo 65). There was additional coal loss north of the pier but no coal loss south of the pier. The net loss of material for this plan was approximately 7 percent. The shoreline, however, continued to accrete. The net increase in beach berm width was approximately 125 ft.

62. An impervious sill was installed (Plan 4A) and tested to determine the amount of coal passing through the voids. The only change noted was a reduction in loss of material to 5 percent, indicating that tracer material was moving over as well as through the voids of the sill.

63. Groin 2 was removed from Plan 4A (Plan 4B) to study its effect on littoral processes. The result was an increased buildup of water against groin 1 which increased the magnitude of the rip currents at this point and sent more material out over the sill. Approximately 8 percent of the material was lost from the study area compared with 5 percent for Plan 4A and 7 percent for Plan 4.

64. In an attempt to eliminate the buildup of water against groin 1 and reduce the resulting rip currents, groin 1 also was removed (Plan 4C). The result was a reduction of rip current magnitude and the formation of a longshore current which reduced loss of the tracer material to about 6 percent.

65. Current patterns and magnitudes for Plan 5 (Photos 66 and 67) for the S60°W deepwater direction revealed the formation of rip currents in most of the breakwater gaps, with an especially strong rip in the gap adjacent to groin 1. Velocities ran as high as 3.5 fps. There was no predominantly strong longshore current.

66. Tracer patterns for Plan 5 are shown in Photos 68 and 69. An appreciable quantity of tracer material left the study area through the breakwater gaps for the large waves, particularly at mhhw. Some tracer material also left the study area for the smaller waves. Once outside the breakwaters, the tracer material migrated to the north past groin 1.

67. Wave heights for Plan 5A are presented in Table 10. A maximum wave height of 8.0 ft was recorded for gage 5 (Plate 7) at mhhw. For this plan, the 7-sec, 11-ft wave at mhhw tended to break seaward of the structure whereas the 14-sec, 7-ft wave broke at the structure. At mllw, both of these waves broke seaward of the structure.

68. Current patterns and magnitudes for Plan 5A (Photos 70 and 71) showed rip currents forming behind many of the breakwater sections. These rips were then deflected by the breakwater where part flowed seaward over the sill and part turned back shoreward to form an eddy. The maximum rip velocity was 2.5 fps compared with 3.5 fps for Plan 5. In the southern quarter of the study area, a buildup of water behind the breakwaters caused a south longshore current to form.

69. Tracer patterns for Plan 5A are shown in Photos 72 and 73.

The only loss of tracer material was very slight and occurred at the southern end of the study area. This was due to the longshore current mentioned above.

70. Semimovable-bed tracer tests were performed for Plan 5A (Photos 74 and 75) using test waves listed in paragraph 37. For each wave, tracer material moved onshore with the exception of small amounts which seeped through the voids of the sill. Rip currents north of the pier created by the 14-sec, 7-ft waves caused some material to migrate out to the sill and through the voids. The percentage of tracer material lost from the study area was approximately 3 percent. However, material was transported shoreward and accretion occurred for each wave tested. In most instances, maximum accretion occurred in the lee of the high-sill structures while minimum accretion occurred in the lee of the low-sill structures. The increase in beach berm width averaged about 100 ft.

71. Wave heights for Plan 5B are presented in Table 11. A maximum wave height of 8.7 ft occurred at gage 2 at mhhw. The higher elevation of the structure effectively prevented overtopping and reduced transmission, as can be seen by comparing wave heights behind the breakwater for Plans 5A and 5B. However, by eliminating the sill between breakwaters, some waves that were tripped by the sill now passed through unobstructed.

72. Current patterns and magnitudes for Plan 5B are shown in Photos 76 and 77. There was no predominant longshore current. Rip currents formed behind most of the structures and exited through the gaps. Velocities were as high as 3.5 fps.

73. Tracer patterns for Plan 5B (Photos 78 and 79) showed a slight loss of material for the large waves. The tracer material was caught in the rips and carried out through the breakwater gaps. Some material also was lost at the southern extremity of the study area.

74. Current pattern and magnitude tests were performed for the Authorized Groin Plan (Photo 80) using test waves listed in paragraph 34. In general, rip currents formed along the side of each groin. Current velocities were as high as 4.3 fps.

75. Tracer test results for the Authorized Groin Plan (Photos 81

and 82) showed that for the 4-ft waves, tracer movement was generally onshore. For the large waves, however, the tracer material migrated north until it entered a rip current. It then moved around the groins to form a bar located at approximately the -10.0 ft contour where it continued its northward movement. Test results indicate that none of the groins of the Authorized Plan trapped tracer material for the larger waves.

76. Current patterns and magnitude tests for Plan 6 (Photo 83) revealed large current velocities apparently unrestricted by the presence of the sheet-pile groins. Velocities were as high as 4.3 fps.

77. Tracer patterns for Plan 6 (Photos 84 and 85) showed substantial loss of material for the large waves. For the 7-sec, 11-ft wave, rip currents carried tracer material offshore to about the -10 ft contour to form a bar where it rapidly migrated northward past groin 1.

78. Current patterns and magnitudes for Plan 7 (Photo 86) were generally quite consistent. Rip currents formed along each side of each groin; and in some cases, eddies formed adjacent to the groins. Current velocities were as high as 4.3 fps.

79. Tracer patterns for Plan 7 (Photos 87 and 88) showed a general shoreward migration. For the large waves, some tracer material moved into the rip currents and out to the groin heads where it quickly settled as the rips dissipated. Little or no tracer material moved from one cell to another and this plan appeared to stabilize the beach.

Discussion of test results

80. A comparison of model current pattern photographs for existing conditions with aerial photographs of the Imperial Beach area indicates that the model accurately reproduced prototype rip and longshore currents. A typical example of this is illustrated in Figures 5 and 6. Figure 5 shows two views of the Imperial Beach area (taken on the same date but at slightly different scales) with waves approaching from the west-northwest direction. Based on (a) an average wavelength of about 200 ft at the end of the pier (about the 20-ft contour), (b) an average tide level of about +2.7 ft mllw, and (c) waves breaking at about the 8-ft contour, the average incident wave was estimated from Figure 5 to

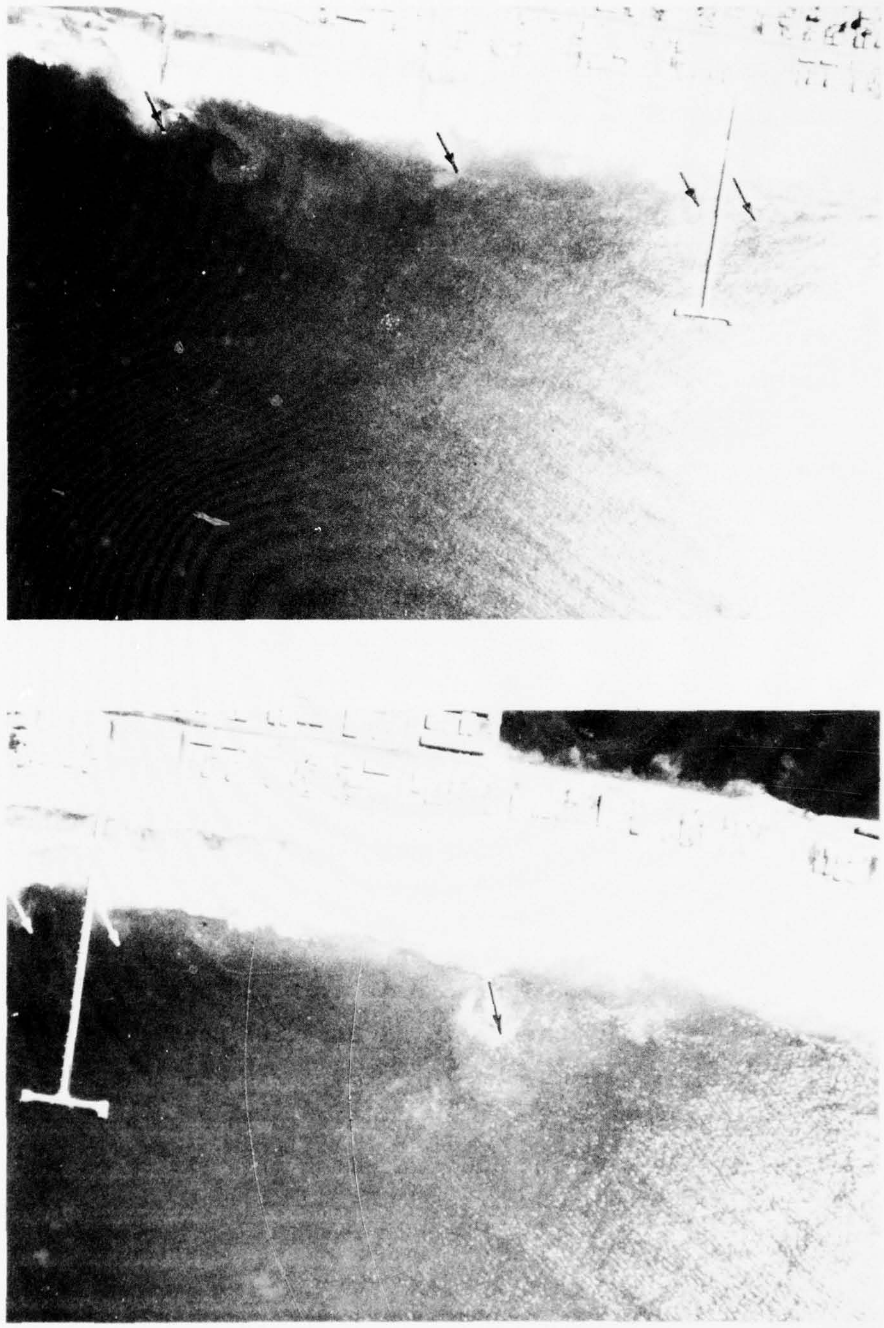

Figure 5. Prototype rip currents; estimated wave period 6 to 9 sec, estimated wave height 6 to 10 ft

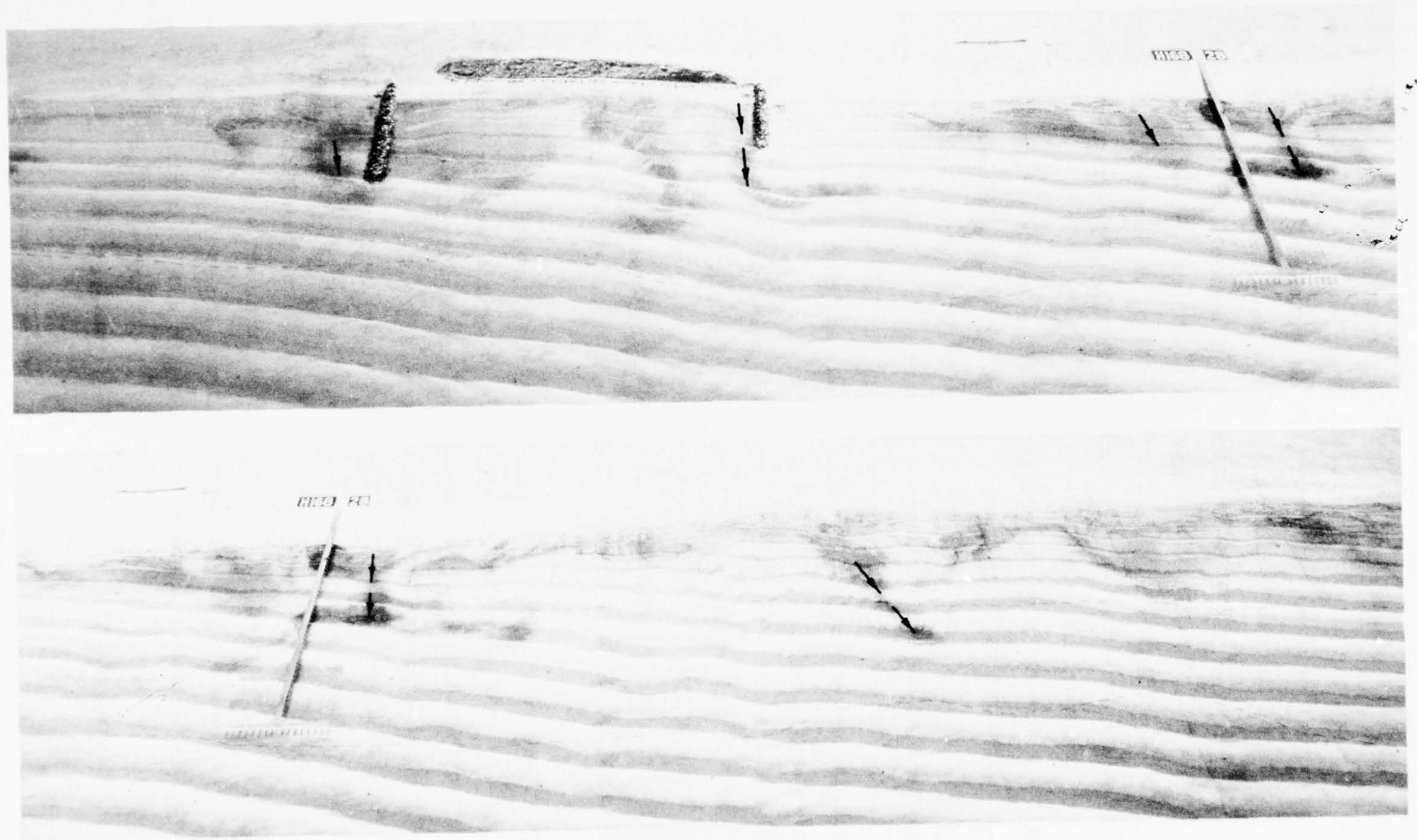

Figure 6. Model rip currents; 7-sec, 7-ft waves from WNW

have a period of between 6 and 9 sec and a height of between 4 and 8 ft. This wave is approximated in the model by a 7-sec, 7-ft wave (at the wave generator) from the west-northwest in Figure 6. The rip current patterns in the two cases (illustrated by cloudy water in Figure 5 and dye in Figure 6 and accented by arrows in both cases) are quite similar. Rip currents are present at both groins, on either side of the pier, and about 1500 to 2000 ft south of the pier. Also, the heads of the rips in both cases are deflected to the south, indicating a probable southward longshore transport of beach material.

81. Test results for existing conditions indicate longshore currents to the north or to the south (depending on incident wave direction) interrupted at regular intervals by strong rip currents. These rips probably transport significant quantitities of sand offshore where it is either (a) transported alongshore on the bar (winter profile), (b) lost in deep water, or (c) transported back shoreward by low-steepness waves.

82. The failure of the two existing groins to trap sand is vividly illustrated in the model by the formation of strong rip currents adjacent to both groins for almost all incident wave conditions. Tracer material was transported around both groins by these rips and moved to the north or south (away from the study area) depending on incident wave direction.

83. Tests with all five authorized groins installed reveal similar problems. For the groins to be effective, they all must be extended a significant distance seaward (well beyond the breaker zone).

84. Since offshore breakwaters (parallel to shore) should be the most effective means of reducing the predominant onshore-offshore movement of sand, such structures were proposed by the SPL as an alternative to the groins at Imperial Beach. The originally proposed breakwater scheme (Plan 1), however, did not reduce incident wave energy enough to preclude continued longshore transport around the groins and loss of material in rip currents. Increasing the length and/or raising the crown elevation of the Plan 1 structures (Plans 1A-1E) met with increasing effectiveness in retaining tracer material. However, due to the

significantly larger volumes of rock required (Table 12), these plans were not considered to be viable alternatives from an economic standpoint.

85. Plan 2 (shorter breakwaters and shorter gaps producing more and weaker rip currents) appeared to retain most of the tracer, but again this plan required a large volume of rock (Table 12).

86. Since model test results had not produced a satisfactory structure at the -15 ft contour location, the only alternative was to move shoreward into shallower water. Plan 3 (located at the -10 ft contour with a 0.0-ft crown elevation) met with partial success, and Plan 3A (adding a groin at the south end to prevent the formation of strong southerly currents) resulted in only very minimal loss of tracer (through the voids of the structure). It was found that submerged breakwaters separated by gaps were not effective at the -10 ft contour. Almost all waves broke over the breakwaters, piling up water between the breakwaters and shore. Since the waves were not breaking in the gaps, this was the path of least resistance for seaward return flow and strong rip currents formed with resulting loss of tracer material from the study area.

87. In an effort to reduce the volume of rock required while keeping rip current magnitudes at a reasonable level, breakwaters with low sills in the gaps were proposed (Plans 4 and 4A). Both these plans were successful in retaining all but small quantities of tracer with Plan 4A (impervious low sills) being slightly better.

88. Tests with structures located at the -5 ft contour revealed that gaps would not be allowable unless the structures were nonovertopping (Plan 5B). Plan 5A appears to be a viable solution and the one which would have the least impact on longshore transport seaward of the structures.

89. Plan 6 (2-ft-high groins) was totally ineffective in trapping tracer material. Plan 7 (nine rubble-mound groins extending to the -12 ft contour with crown elevations and cross sections, shown in Plate 12, similar to the existing groins) was very effective in trapping tracer material; but the volume of rock required (Table 12) was

significantly greater than the best offshore breakwater plans.

90. Results of semimovable-bed tests were useful and interesting; however, caution must be exercised in their interpretation. One basic restriction was that erosion was limited to 1/2 in. (approximately 3 ft prototype) by the concrete model floor. For steep waves, material in the breaker zone eroded and increased the water depth (and thereby the breaker height possible) until the model floor was uncovered. The presence of the fixed-bed model floor then caused the breaking point to remain stationary since no deeper erosion was possible. This limited the steepness of the beach and thereby limited the amount of beach erosion possible. The movement of tracer material also was very sensitive to wave size and order of testing. Regardless of the shortcomings of these tests, however, relative comparisons between plans for like test conditions should be valid; and the losses of 3, 5, and 14 percent for Plan 5A, Plan 4A, and existing conditions, respectively, indicate the value of the offshore breakwaters in reducing erosion. These comparisons are even more in favor of Plans 5A and 4A when considered in light of the fact that losses would probably continue with time for existing conditions as wave conditions change (i.e., direction, height, period, etc.); whereas the values for the offshore breakwaters are probably limiting values which, once lost initially, would not be lost again due to the partially controlled wave environment inside the structures.

91. The stated function of the proposed structures at Imperial Beach is the protection and retention of existing (or artificially placed) beaches. While it is not intended that the beaches be widened by the entrapment of longshore transport, some of the structures will be more effective in doing this with the potential of erosion downdrift and/or updrift from the present study area. Therefore, it would appear that a plan which retains the proposed fill at Imperial Beach yet minimizes effects on adjacent beaches (Plan 5A) would be the optimum. However, this requisite requirement must be weighed with construction costs, material availability, aesthetics, and wishes of local people; and another of the functionally acceptable plans (3A, 4A, or 5B) may be selected. During preparation of this report (after completion of

model testing), it was proposed that the 7000-ft-long Plan 4A structure (five 700-ft-long breakwaters and five 700-ft-long low sills) be modified to consist of four breakwaters and three low sills, each 715 ft long, for a total length of 5000 ft, extending southward from groin 1. The southern groin also would be moved to the southern tip of the structure (now a breakwater instead of a low sill) instead of forming the "T-head" arrangement shown for Plan 4A. This revised configuration should function very much the same as Plan 4A for the 5000 ft being protected. However, this leaves 2000 ft of the original study area unprotected. Moving the south groin to the south end of the structure should provide additional benefits outside the study area by extending the fillet formed by this groin farther southward.

92. Longshore transport rates at Imperial Beach have been estimated to be relatively small (Reference 16) compared with other areas in southern California, and it is known that the Tijuana Shoal is depleted of sand and composed primarily of cobbles (Inman[1]). In addition, the southern end of Imperial Beach becomes cobbles during certain times of the winter when the sand is eroded (both offshore and alongshore). Therefore, little sand is available to be transported away from the shoal; and even if the planned structures do trap a small amount of sand traveling to the north, any increased erosion to the north of the protected area would have occurred anyway in a relatively short period of time. The southern portion of Imperial Beach has practically no sand on it and since the Tijuana Shoal is composed of cobbles, proposed structures will only have a minimal effect on this area. Most waves approach the beach very nearly normal and the majority of the transport appears to be onshore and offshore transport rather than alongshore. Therefore, in this particular case it may be justifiable to select Plans 3A, 4A, or 5B (or a slight modification of one of these) rather than Plan 5A which would normally be the recommended plan.

PART V: CONCLUSIONS

92. Based on results of the three-dimensional model investigation reported herein, it is concluded that:

a. Existing conditions are characterized by strong rip currents and longshore currents for most wave conditions with considerable onshore-offshore movement of sand.

b. The original improvement plan for Imperial Beach (i.e., the authorized five-groin plan) is ineffective in trapping tracer material and, in fact, contributes to the offshore movement.

c. The originally proposed breakwater plan (Plan 1) is not adequate for full protection of the beach. Raising the crown elevation of the Plan 1 structure to a sufficient height will make this plan effective from a functional standpoint; however, a substantial cost increase will result due to greatly increased volume of work required.

d. Of the plans tested, Plans 3A, 4A, 5A, 5B, and 7 all provide adequate beach protection. Plans 4A and 5A require the least volume of rock but may pose some construction problems. Plan 5A would have the least impact on the longshore transport (due to its location at the -5.0 ft contour resulting in more natural sand bypassing around the structure) and therefore would be the most viable plan from the viewpoint of minimizing effect of the improvement on adjacent shores. The Plan 3A and 4A breakwaters would be submerged except for extremely low tide stages and, therefore, would probably be the most aesthetically pleasing.

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Table 1

Estimated Duration and Magnitude of Deepwater Waves

Approaching Imperial Beach from Various Directions

Wave Height* ft	Duration, hr/yr, per Wave Period,* Sec 2-4	4-6	6-8	≥8	8-10	10-12	12-14	14-16	16-18	≥18	Total
					Northwest						
1-2	193										193
2-3		105									105
3-4		79									79
4-5		44									44
5-6			18								18
6-8			9								9
8-10			5								5
10-12				5							5
12-14				5							5
Total	193	228	32	10							463
					West-Northwest						
1-2	61										61
2-3		35									35
3-4		26									26
4-5		18									18
5-6			9								9
6-8			5								5
8-10			5								5
10-12				5							5
12-14											
Total	61	79	19	5							164
					West						
0-1	167		27		44	14					252
1-2	131		5		9	44	26	9			224
2-3		70				9	44	27	14		164
3-4		88				5	10	9			112
4-5		44					10	9	9		72
5-6			18				5		9		32
6-8			53			9		10	18	5	95
8-10			9						9		18
10-12					5					5	10
12-14					5						5
Total	298	202	107		58	81	95	64	59	10	984

(Continued)

* Wave-height and wave-period groupings include the lower but not the upper values.

Table 1 (Concluded)

Wave Height* ft	2-4	4-6	6-8	≥8	8-10	10-12	12-14	14-16	16-18	≥18	Total
	Duration, hr/yr, per Wave Period, Sec										
					West-Southwest						
0-1			9		26	9	26	9	9		88
1-2	26	26	14		9	53	44	79	18	5	274
2-3		9	15		9	5	18	35	26	5	112
3-4		5	18		9		5	9	44	5	95
4-5		5					9	26	9	9	58
5-6			5		5	5	5	5	18	5	48
6-8			5					5		9	19
Total	26	45	56		58	72	107	168	124	38	694
					Southwest						
0-1	412	412	417		417	412	123	412	5		2610
1-2	123	35	44		23	9	491	359	35	9	1128
2-3					5		360	289	61	9	724
3-4			18			5	70	88	53	9	243
4-5											0
5-6			5								5
6-8			5								5
8-10			5							5	10
Total	535	447	494		445	426	1044	1148	154	32	4725
					South-Southwest						
0-1	61	61	61				219	105	18	23	548
1-2	18	18	5		9	9	412	26	44	5	546
2-3		9				18	105	105	35	14	286
3-4		9	5				9	35	35	18	111
4-5		5									5
5-6			5								5
6-8			5								5
8-10			5								5
Total	79	102	91		9	27	745	271	132	60	1511
					South						
0-1	281	281	281		290	290	377	281	123	44	2248
1-2	53	61	5		9	9	666	640	263	61	1767
2-3		26	9			26	105	79	18	5	268
3-4		18			5	5	14	18	14		74
4-5		9									9
5-6			9			5					14
6-8			5								5
8-10			5								5
Total	334	395	314		304	335	1162	1018	418	109	4390

Table 2

Estimated Duration and Magnitude of Shallow-Water Waves Approaching Imperial Beach from Various Directions

Wave Height* ft	Duration, hr/yr, per Wave Period,* sec 2-4	4-6	6-8	8-10	10-12	12-14	14-16	≥16	Total
N60°W									
1-3	254	245							499
3-5		62	41						103
5-7			10						10
7-9				15					15
9									
10-12									
≥12									
Total	254	307	51	15					627
West									
0-3	298	70	32	53	67	26	9		555
3-5		132			5	54	36	13	240
5-7			18			15	9	9	51
7-9			53		9		10	9	81
9-11			9					23	32
11-13				5				9	14
13-15				5				5	10
Total	298	202	112	63	81	95	64	68	983
S60°W									
0-3	561	482	489	489	483	1062	1183	172	4926
3-5		10	36	9	10	84	123	129	401
5-7			10	5	5	5	10	23	58
7-9			10					9	19
9-11			5					5	10
Total	561	492	550	503	498	1151	1316	343	5414

* Wave-height and wave period groupings include the lower but not the upper values.

Table 3

Wave Heights for Existing Conditions

swl = +5.4 ft = mhhw

Test Wave (at Generator)				Wave Heights, ft, for Various Test Directions														
Period sec	Height ft	K_f*	Adjusted Height** ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15
						West Deepwater Direction												
7	4	1.123	3.6	2.5	3.0	2.1	3.2	3.0	2.6	5.3	3.8	4.1	4.6	4.1	3.6	2.0	1.8	1.7
7	11	1.123	9.8	8.9	8.2	6.0	8.0	7.5	5.3	8.8	8.1	6.9	8.2	10.0	9.8	6.9	5.2	4.6
10	4	1.131	3.5	5.8	5.2	4.2	4.9	4.7	3.7	5.7	4.3	4.4	7.7	5.9	6.5	2.9	2.4	2.2
10	15	1.131	13.3	9.8	17.4	17.0	10.7	12.4	13.2	11.4	14.1	13.8	10.5	13.6	15.3	7.4	13.7	12.2
14	4	1.118	3.6	5.3	4.2	4.0	8.1	4.3	4.9	8.0	4.9	5.8	10.5	7.0	7.2	4.1	3.5	3.1
14	9	1.118	8.1	15.9	14.7	14.4	9.7	15.2	13.7	8.8	14.8	17.1	9.6	15.9	17.2	7.0	10.0	9.0
18	4	1.107	3.6	5.5	4.9	3.7	8.2	5.8	5.4	8.0	6.2	6.0	7.6	6.1	5.4	3.1	2.0	2.4
18	15	1.107	13.6	15.1	15.4	17.9	10.8	14.2	16.0	11.1	14.6	17.4	8.2	16.8	15.2	8.6	13.6	15.1
						$N60^{o}W$ Deepwater Direction												
7	4	1.128	3.5	4.8	4.4	4.5	3.6	3.5	2.8	3.4	2.7	3.6	4.0	4.0	4.4	5.3	4.3	4.3
7	7	1.128	6.2	7.0	6.4	6.3	6.1	5.6	5.4	6.9	5.2	6.0	7.1	7.1	7.7	7.9	6.0	6.6
10	4	1.136	3.5	5.5	5.1	4.1	4.3	3.8	3.9	4.8	3.1	4.3	7.9	5.2	6.4	7.2	4.7	4.8
10	9	1.136	7.9	13.9	14.2	11.0	9.4	12.9	10.9	11.2	13.5	11.2	8.1	14.4	14.1	6.9	12.4	11.5
						$S60^{o}W$ Deepwater Direction												
7	4	1.151	3.5	2.4	2.8	2.7	3.2	2.8	2.8	5.6	4.3	5.2	3.7	3.1	3.9	4.0	3.4	2.7
7	11	1.151	9.6	12.0	9.2	10.5	10.5	8.9	10.0	8.3	11.7	12.8	11.3	11.7	12.2	8.1	8.3	8.4
10	4	1.154	3.5	4.5	4.6	3.5	5.4	4.4	3.2	8.6	5.7	6.4	6.9	4.3	5.5	4.8	3.9	3.5
10	7	1.154	6.1	10.5	7.6	5.5	11.8	8.2	7.7	13.3	11.5	11.3	10.7	8.5	9.7	9.7	9.2	7.7
14	4	1.137	3.5	6.1	4.9	4.6	6.4	3.8	4.5	10.6	7.0	8.3	6.0	5.0	4.6	8.0	4.9	5.6
14	7	1.137	6.2	11.6	9.4	8.8	13.7	9.6	9.0	16.6	12.6	13.3	12.5	9.3	8.2	7.7	7.9	8.5
18	4	1.123	3.6	5.1	4.6	4.3	5.5	4.6	4.1	10.4	7.8	7.1	5.3	4.6	3.7	7.5	4.7	5.1
18	11	1.123	9.8	16.9	15.1	14.2	10.4	15.7	14.4	10.2	12.9	16.1	8.1	18.0	16.6	8.9	13.8	13.5

* K_f = factor to correct for wave attenuation due to viscous friction at the model bottom.
** Adjusted height = initial height divided by attenuation factor (K_f).

Table 4

Wave Heights for Existing Conditions

swl = 0.0 ft = mllw

Test Wave (at Generator)				Wave Heights, ft, for Various Test Directions														
Period sec	Height ft	K_f*	Adjusted Height** ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11	Gage 12	Gage 13	Gage 14	Gage 15
				West Deepwater Direction														
7	4	1.165	3.4	3.4	4.0	2.8	4.4	3.8	3.1	6.1	4.1	5.0	4.5	5.1	5.0	3.4	2.5	1.9
7	11	1.165	9.4	5.6	12.5	10.0	4.4	11.8	11.0	5.1	11.0	7.8	3.8	11.5	12.4	2.8	7.4	7.7
14	4	1.148	3.5	4.3	3.6	3.1	7.1	3.9	3.5	6.6	3.5	4.3	5.5	5.0	4.9	3.7	2.1	2.0
14	9	1.148	7.8	10.5	13.1	10.8	6.2	13.4	11.0	6.1	13.0	9.7	6.0	13.1	13.8	3.3	7.1	7.3
				$N60^{o}W$ Deepwater Direction														
7	4	1.164	3.4	5.5	4.8	4.5	4.9	4.1	3.1	5.0	2.6	3.8	5.5	5.0	5.4	2.8	3.7	5.0
7	7	1.164	6.0	7.4	8.6	6.5	4.9	6.6	6.6	7.5	7.0	7.2	4.1	8.6	8.6	3.0	7.3	7.5
10	4	1.164	3.4	7.1	5.7	3.9	6.1	4.0	4.0	5.6	2.8	4.8	5.2	5.5	7.0	3.3	5.2	4.0
10	9	1.164	7.7	6.5	13.5	11.6	4.6	12.7	10.2	5.5	11.5	11.7	4.5	13.3	12.8	3.3	12.9	10.7
				$S60^{o}W$ Deepwater Direction														
7	4	1.205	3.3	2.4	2.7	2.8	4.2	3.1	2.9	6.9	4.4	5.8	4.5	2.5	4.4	3.2	3.0	2.9
7	11	1.205	9.1	8.0	8.8	10.3	4.8	10.4	10.8	6.1	8.6	10.6	3.8	12.3	11.6	3.1	11.0	9.9
14	4	1.175	3.4	5.7	4.5	3.7	5.7	4.3	2.9	7.3	6.3	7.0	5.5	4.3	4.4	3.3	6.1	4.8
14	7	1.175	6.0	8.9	5.9	5.1	7.7	5.7	3.8	5.9	8.8	9.9	5.2	6.2	5.3	3.8	8.3	7.0

*K_f = factor to correct for wave attenuation due to viscous friction at the model bottom.
** Adjusted height = initial height divided by attenuation factor (K_f).

Table 5

Wave Heights for Plan 1

Test Wave (at Generator)				Wave Heights, ft, for S60°W Deepwater Direction										
Period sec	Height ft	K_f*	Adjusted Height ft**	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11
								swl = +5.4 ft = mhhw						
7	4	1.151	3.5	5.2	2.0	5.8	3.1	3.8	5.0	5.6	3.1	4.9	2.7	3.2
7	11	1.151	9.6	5.0	5.8	4.2	7.3	4.3	4.9	5.9	5.7	4.1	3.8	2.7
10	4	1.154	3.5	6.1	2.5	9.8	4.0	6.2	5.6	9.5	5.8	5.7	3.0	5.1
10	7	1.154	6.1	11.5	9.7	9.7	9.5	11.5	11.6	14.9	11.9	9.0	8.5	8.1
14	4	1.137	3.5	6.1	5.0	8.8	5.7	5.8	7.8	9.2	7.3	8.5	5.3	7.6
14	7	1.137	6.2	10.4	10.8	13.2	11.1	14.7	10.9	17.1	12.9	11.4	9.3	8.2
18	4	1.123	3.6	5.1	5.5	6.8	5.7	4.8	7.9	7.0	5.9	5.8	5.3	5.9
18	11	1.123	9.8	14.0	16.1	8.5	10.3	8.6	9.2	11.6	9.7	10.0	7.2	7.0
								swl = 0.0 ft = mllw						
7	4	1.205	3.3	5.2	2.0	5.8	3.1	3.8	5.0	5.6	3.1	4.9	2.7	3.2
7	11	1.205	9.1	5.0	5.8	4.2	7.3	4.3	4.9	5.9	5.7	4.1	3.8	2.7
14	4	1.175	3.4	5.4	4.0	7.3	2.9	5.1	6.0	7.7	5.7	5.0	4.9	2.5
14	7	1.175	6.0	6.1	6.0	7.0	6.0	7.4	7.6	6.9	6.7	4.9	4.2	2.5

*K_f = factor to correct for wave attenuation due to viscous friction at the model bottom

**Adjusted Height = initial height divided by attenuation factor (K_f)

Table 6

Wave Heights for Plan 2

Test Wave (at Generator)				Wave Heights, ft, for S60°W Deepwater Direction										
Period sec	Height ft	K_f*	Adjusted Height ft**	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11
								swl = +5.4 ft = mhhw						
7	4	1.151	3.5	2.3	3.9	2.7	3.1	3.6	5.6	7.3	3.7	3.2	3.4	2.6
7	11	1.151	9.6	5.6	9.7	8.2	11.9	10.8	12.6	11.9	8.1	8.8	11.6	8.7
10	4	1.154	3.5	5.7	4.6	4.1	5.1	4.4	8.7	10.8	6.8	5.9	6.1	6.5
10	7	1.154	6.1	10.2	7.0	8.6	13.7	6.6	13.5	15.9	10.6	11.4	12.8	10.7
14	4	1.137	3.5	7.9	7.2	5.7	5.8	4.3	9.3	12.1	7.9	5.6	8.0	7.9
14	7	1.137	6.2	11.7	12.4	10.5	10.6	9.7	11.0	17.0	14.6	11.8	13.6	12.1
								swl = 0.0 ft = mllw						
7	4	1.205	3.3	1.4	3.6	2.1	4.2	2.5	2.9	7.8	3.7	3.2	5.3	2.5
7	11	1.205	9.1	7.8	6.6	8.9	5.3	6.3	7.2	6.1	6.6	5.5	5.3	5.4
14	4	1.175	3.4	3.9	4.8	2.8	5.7	4.1	4.9	7.7	6.2	6.4	7.9	6.8
14	7	1.175	6.0	4.6	5.8	3.6	7.0	3.8	6.8	6.4	8.2	7.7	5.8	8.8

* K_f = factor to correct for wave attenuation due to viscous friction at the model bottom

** Adjusted Height = initial height divided by attenuation factor (K_f)

Table 7

Wave Heights for Plan 3A

Test Wave (at Generator)				Wave Heights, ft, for S60°W Deepwater Direction										
Period sec	Height ft	K_f*	Adjusted Height ft **	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11
				swl = +5.4 ft = mhhw										
7	4	1.151	3.5	3.0	4.4	2.9	2.5	3.3	4.3	6.2	4.0	3.6	3.8	2.9
7	11	1.151	9.6	7.7	7.7	7.0	6.9	7.3	5.9	7.8	7.3	7.6	7.2	6.1
10	4	1.154	3.5	5.1	8.0	5.0	3.7	5.2	7.3	7.6	7.1	5.9	6.6	6.6
10	7	1.154	6.1	10.9	7.9	7.5	9.0	8.4	6.4	9.3	7.4	8.1	7.9	7.0
14	4	1.137	3.5	7.0	7.9	5.7	6.8	5.4	9.1	9.7	8.4	6.4	8.4	5.9
14	7	1.137	6.2	12.9	10.0	8.5	6.9	8.7	7.3	9.8	8.5	10.5	7.6	6.7
18	4	1.123	3.6	3.4	4.3	5.0	4.1	3.3	6.9	9.7	8.9	5.1	7.1	5.7
18	11	1.123	9.8	12.7	9.0	8.6	9.5	9.4	7.9	9.7	8.9	10.0	6.9	7.4
				swl = 0.0 ft = mllw										
7	4	1.205	3.3	2.1	2.9	1.8	1.6	2.1	3.2	3.5	2.9	2.4	2.7	0.6
7	11	1.205	9.1	5.5	5.1	4.5	4.1	3.3	4.2	3.7	4.5	3.9	4.4	2.4
14	4	1.175	3.4	5.1	5.1	2.9	2.1	3.7	4.7	5.2	5.5	4.9	4.6	3.3
14	7	1.175	6.0	6.1	6.1	3.2	3.6	3.4	5.2	5.3	5.8	4.9	5.1	3.6

* K_f = factor to correct for wave attenuation due to viscous friction at the model bottom.

** Adjusted Height = initial height divided by attenuation factor (K_f).

Table 8

Wave Heights for Plan 4

swl = +5.4 ft = mhhw

Test Wave (at Generator)				Wave Heights, ft, for Various Test Directions										
Period sec	Height ft	Kf*	Adjusted Height** ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11
				N60°W Deepwater Direction										
7	4	1.110	3.6	2.2	3.7	3.6	4.2	3.3	3.6	4.7	4.4	2.1	4.3	4.4
7	7	1.110	6.3	5.7	5.3	6.0	5.5	5.1	6.4	6.3	8.1	5.8	6.2	6.1
10	4	1.115	3.6	3.6	3.6	6.0	4.6	4.6	5.4	5.5	8.2	5.9	5.6	5.3
10	9	1.115	8.1	9.5	8.0	7.2	7.6	7.7	8.9	7.9	7.3	7.0	6.5	6.6
				West Deepwater Direction										
7	4	1.108	3.6	3.6	3.0	4.9	3.1	3.1	4.4	5.5	5.3	2.3	4.7	4.3
7	11	1.108	9.9	8.4	8.9	6.2	8.2	7.0	7.8	7.4	6.4	6.3	5.0	5.0
10	4	1.114	3.6	4.3	2.9	5.8	1.8	4.1	5.2	6.2	7.7	4.7	6.0	4.5
10	15	1.114	13.5	10.8	12.7	9.1	9.1	10.4	9.5	9.2	7.8	7.2	6.3	7.3
14	4	1.103	3.6	5.5	4.6	7.3	6.1	6.7	6.9	8.2	8.1	5.2	6.2	5.3
14	9	1.103	8.2	12.6	14.3	9.0	9.4	9.0	9.1	9.6	8.1	7.7	6.6	6.5
18	4	1.232	3.2	6.3	5.3	6.8	6.5	5.2	7.0	9.8	11.5	6.7	9.3	8.1
18	15	1.232	12.2	11.2	15.7	8.8	10.6	10.9	10.0	11.1	9.8	9.1	8.9	8.3
				S60°W Deepwater Direction										
7	4	1.151	3.5	3.7	2.7	3.1	3.2	4.3	6.5	4.5	5.1	4.0	3.4	3.4
7	11	1.151	9.6	7.3	9.4	7.1	8.8	6.9	8.5	7.6	5.8	5.9	5.5	5.7
10	4	1.154	3.5	6.4	4.0	6.8	4.3	5.6	8.4	6.1	3.7	5.6	4.7	5.8
10	7	1.154	6.1	9.3	7.1	7.9	10.0	9.2	8.8	8.4	7.0	7.2	6.9	9.0
14	4	1.137	3.5	5.1	5.6	6.2	5.0	6.2	9.7	8.1	7.6	7.8	6.5	7.1
14	7	1.137	6.2	10.0	10.6	8.3	8.9	11.4	12.5	10.3	9.4	7.3	6.9	6.9
18	4	1.123	3.6	3.8	3.6	4.4	4.7	5.3	9.2	5.4	7.5	7.0	5.5	4.8
18	11	1.123	9.8	12.1	13.9	9.4	11.0	9.7	9.0	11.8	7.7	9.8	7.0	8.1

*K_f = factor to correct for wave attenuation due to viscous friction at the model bottom.

**Adjusted height = initial height divided by attenuation factor (K_f).

Table 9

Wave Heights for Plan 4

swl = 0.0 ft = mllw

Test Wave (at Generator)				Wave Heights, ft, for Various Test Directions										
Period sec	Height ft	K_f*	Adjusted Height** ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11
				N60°W Deepwater Direction										
7	7	1.144	6.1	3.8	6.3	3.9	4.5	4.3	5.3	4.7	4.3	4.3	3.0	2.8
10	9	1.144	7.9	6.2	8.1	5.5	6.0	5.9	5.9	6.3	4.8	5.1	4.4	3.4
				West Deepwater Direction										
7	4	1.141	3.5	3.4	3.5	4.1	3.5	3.5	4.5	4.3	3.8	2.8	2.9	3.1
7	11	1.141	9.6	6.1	6.9	4.5	4.9	5.4	4.8	5.2	4.7	4.1	3.5	3.1
14	4	1.127	3.5	4.4	5.8	5.6	4.5	4.5	5.8	5.7	4.1	4.2	3.1	3.2
14	9	1.127	8.0	8.4	10.3	5.9	6.0	6.6	7.0	7.1	5.5	5.5	5.2	4.1
				S60°W Deepwater Direction										
7	4	1.205	3.3	4.1	2.6	2.5	2.4	3.4	4.2	2.7	3.3	3.2	2.8	2.6
7	11	1.205	9.1	5.4	6.4	3.6	4.5	4.6	5.1	4.5	5.0	3.5	2.7	2.9
14	4	1.175	3.4	4.8	4.1	3.9	3.2	3.5	5.0	5.2	4.0	4.0	3.1	2.9
14	7	1.175	6.0	5.9	5.7	4.8	4.8	5.1	5.0	5.4	4.2	4.1	3.0	3.2

*K_f = Factor to correct for wave attenuation due to viscous friction at the model bottom.

**Adjusted height = initial height divided by attenuation factor (K_f).

Table 10

Wave Heights for Plan 5A

Test Wave (at Generator)				Wave Heights (ft) for S60°W Deepwater Direction										
Period sec	Height ft	K_f*	Adjusted Height ft**	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11
				swl = +5.4 ft = mhhw										
7	4	1.207	3.3	2.2	5.1	4.5	1.7	3.2	1.9	3.2	3.5	2.6	4.3	2.8
7	11	1.207	9.1	4.9	5.4	5.1	3.8	4.4	2.3	2.6	4.0	2.1	5.1	2.9
14	4	1.177	3.4	4.2	5.4	5.3	4.6	6.2	4.8	4.6	5.1	4.0	5.1	4.4
14	7	1.177	5.9	7.0	6.4	6.2	5.4	8.0	6.5	5.3	5.6	5.7	5.7	5.7
				swl = 0.0 ft = mllw										
7	11	1.261	8.7	1.8	2.2	1.8	0.8	1.6	0.6	0.6	1.1	0.6	1.9	0.6
14	7	1.214	5.8	3.2	3.7	3.0	1.4	3.1	2.3	1.0	1.2	1.2	2.2	1.3

*K_f = Factor to correct for wave attenuation due to viscous friction at the model bottom.

**Adjusted height = initial height divided by attenuation factor (K_f).

Table 11

Wave Heights for Plan 5B

Test Wave (at Generator)				Wave Heights (ft) for S60°W Deepwater Direction										
Period sec	Height ft	K_f*	Adjusted Height ft**	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9	Gage 10	Gage 11
				swl = +5.4 ft = mhhw										
7	4	1.215	3.3	0.9	5.5	4.2	0.8	3.1	1.0	0.9	3.6	0.9	5.5	1.0
7	11	1.215	9.1	1.8	5.3	4.4	2.7	4.1	1.4	1.6	4.6	1.3	4.7	1.2
14	4	1.184	3.4	2.1	8.7	6.4	2.8	6.5	2.7	2.7	5.7	1.9	4.9	2.6
14	7	1.184	5.9	3.3	7.9	5.6	3.1	5.8	3.7	3.5	5.7	3.5	5.0	3.4
				swl = 0.0 ft = mllw										
7	11	1.271	8.7	1.0	2.5	2.0	1.6	2.0	0.5	0.6	1.6	0.4	1.7	0.5
14	7	1.223	5.7	1.1	2.8	2.0	0.6	1.9	1.0	0.5	2.0	0.8	2.8	0.9

*K_f = Factor to correct for wave attenuation due to viscous friction at the model bottom.

**Adjusted height = initial height divided by attenuation factor (K_f).

Table 12

Dimensions of Structures Tested

Breakwater Plans

Plan	Individual Breakwater Length, ft	No. of Breakwater Segments	Length of Gap ft	Length of Sill ft	Depth Contour at Breakwater ft	Crown Elevation, ft Breakwater	Crown Elevation, ft Sill	Volume of Rock x 10^3 yd^3
1	700	5	700	--	-15	-2.0	--	58.1
1A	6300	--	--	--	-15	-2.0	--	104.7
1B	700	5	700	--	-15	+5.4	--	120.6
1C	900	5	500	--	-15	+5.4	--	155.0
1D	1000	5	400	--	-15	+5.4	--	172.3
1E*	1000	5	400	--	-15	+5.4	--	172.3
2	350	10	350	--	-15	+5.4	--	120.6
3	6650	--	--	--	-10	0.0	--	73.9
3A	6650	--	--	--	-10	0.0	--	76.0**
4	700	5	--	700	-10	0.0	-5.0	55.6**
5	350	10	350	--	-5	+5.0	--	38.9
5A	350	10	--	300	-5	+5.0	0.0	50.1
5B	350	10	350	--	-5	+10.0	--	72.9

Groin Plans

Plan	No. of Groins	Average Groin Length, ft	Depth Contour Extended, ft	Volume of Rock x 10^3 yd^3
Authorized Groin Plan	5	450	-6.0	19.2†
Plan 6	9	750	-12.0	--
Plan 7	9	750	-12.0	118.0†

* Same as Plan 1D except all breakwaters were shifted 350 ft northward.
** Includes groin at south end.
† Exclusive of existing groins 1 and 2.

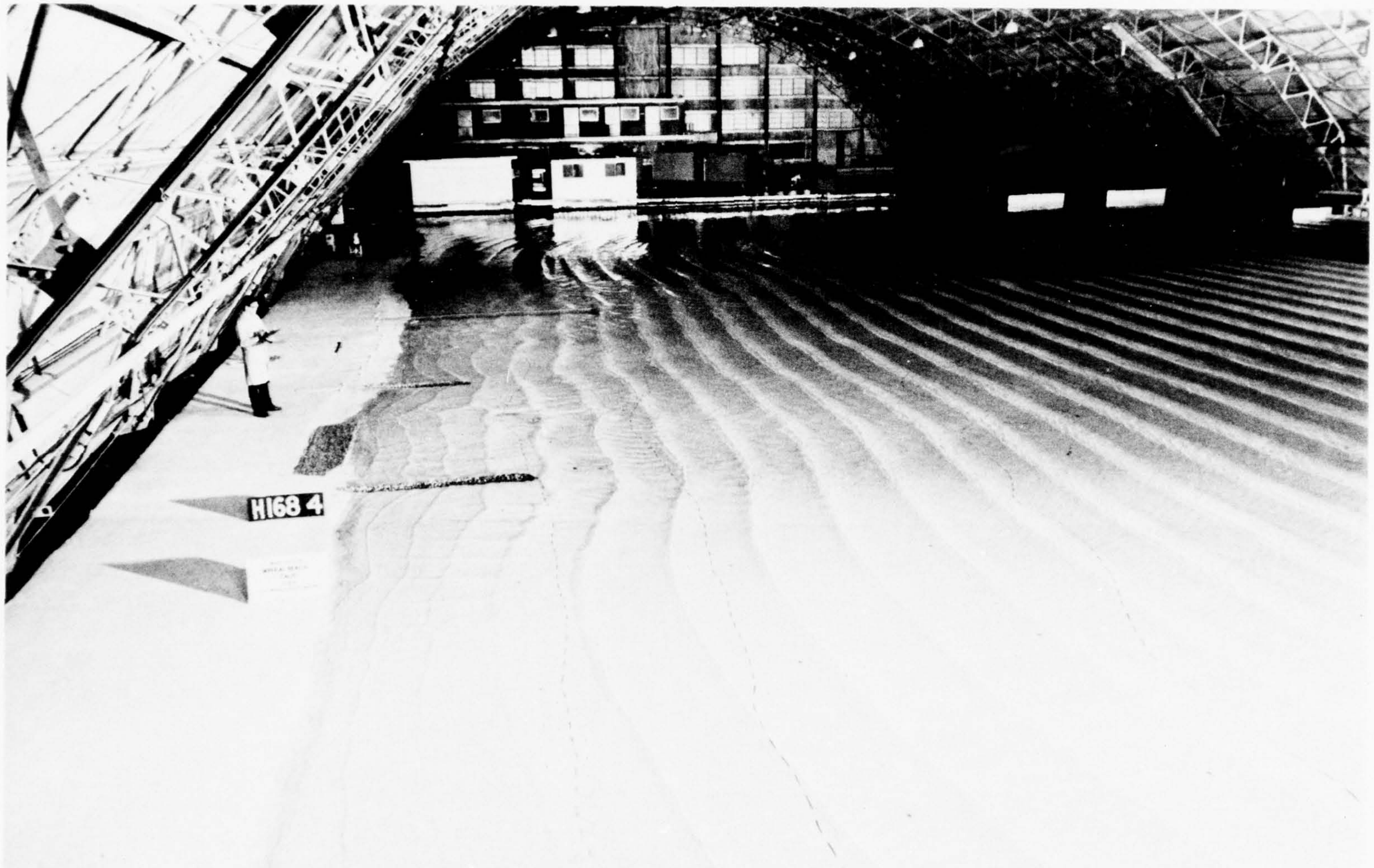
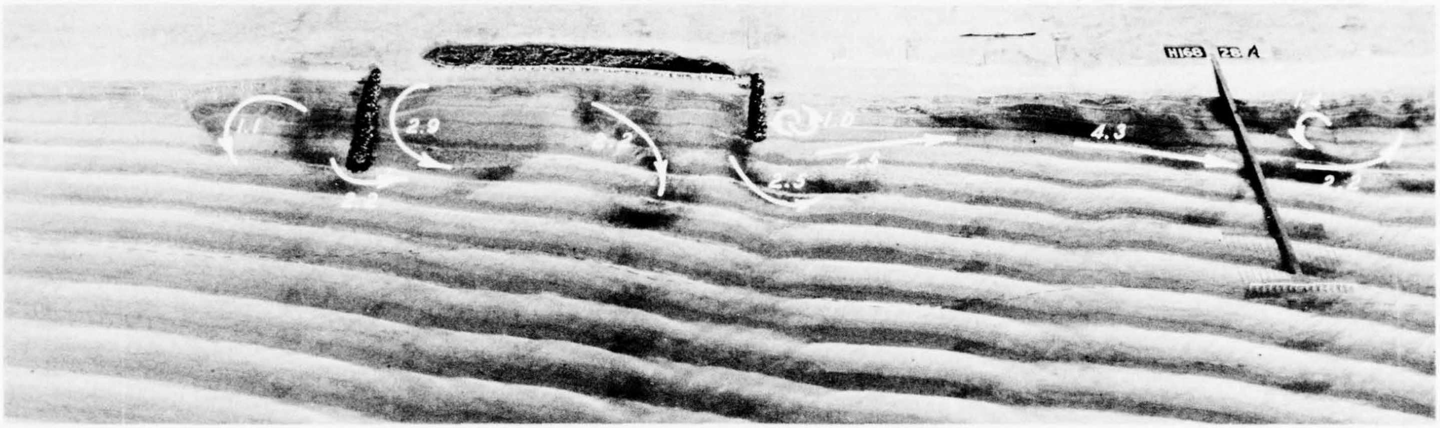


Photo 1. General view of the model



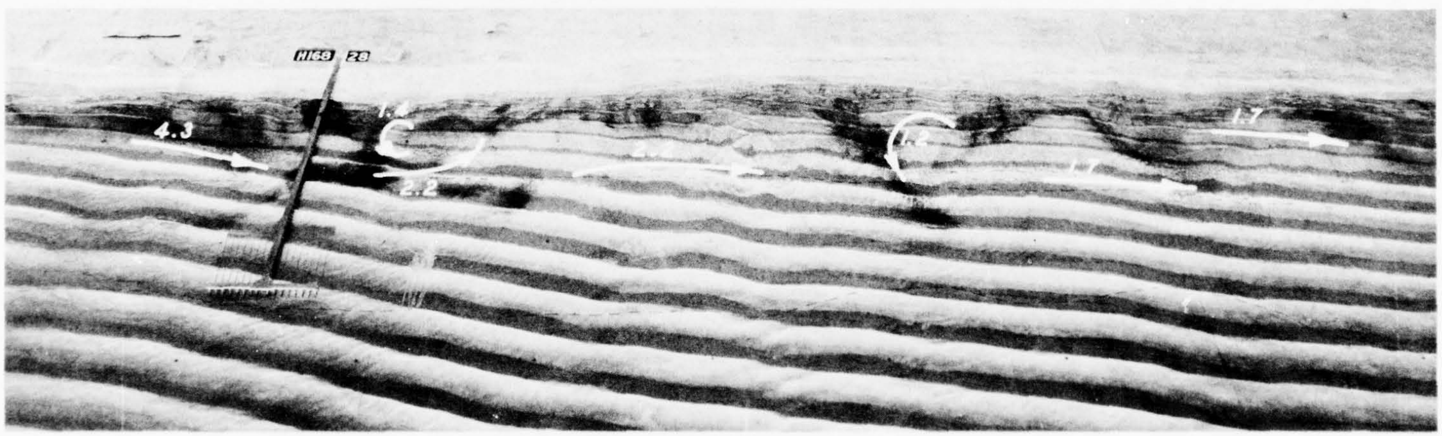


Photo 2. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 7-sec, 7-ft waves from N60°W at mllw

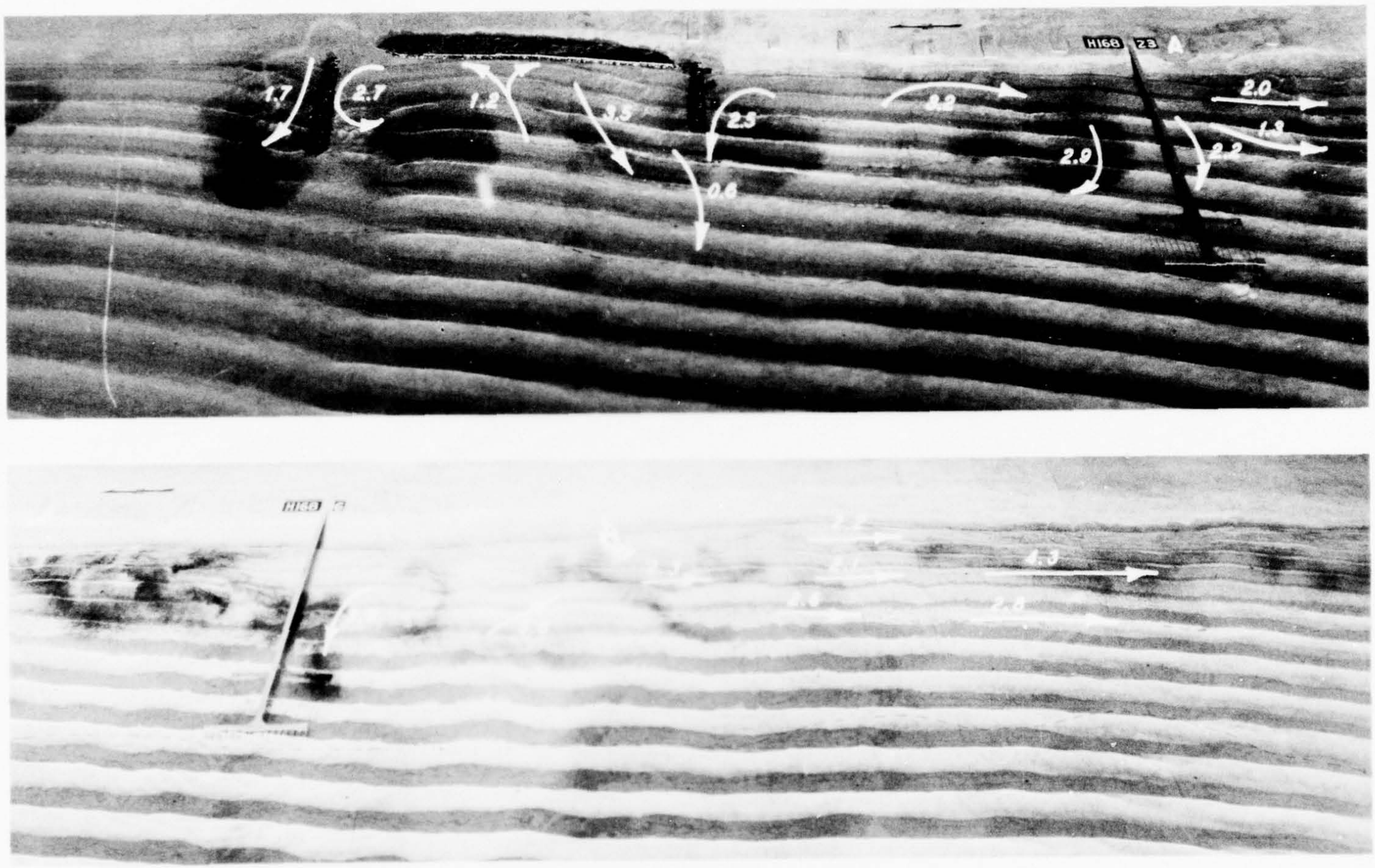
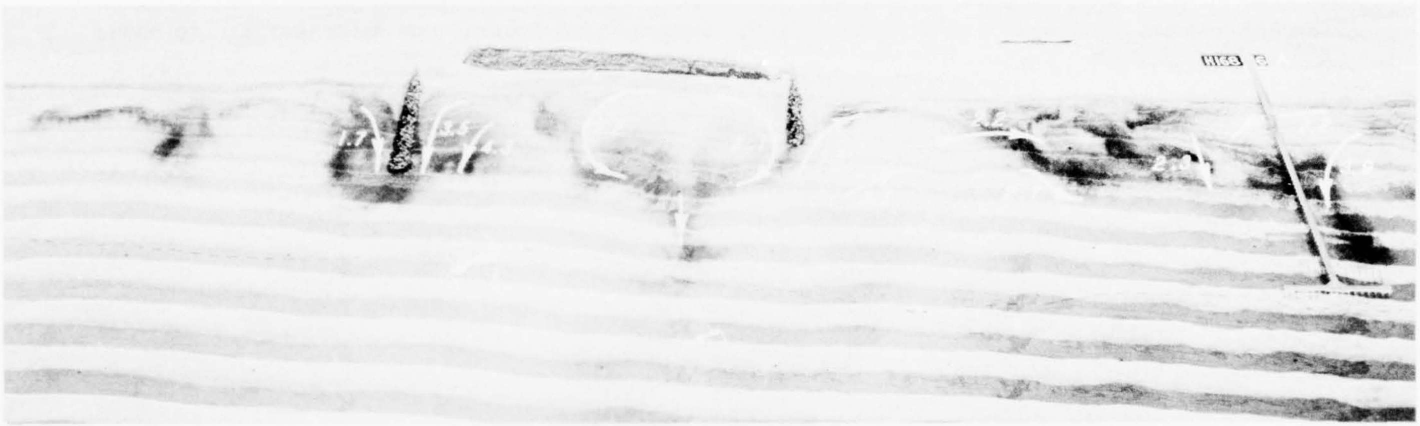


Photo 3. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 7-sec, 7-ft waves from N60°W at mhhw



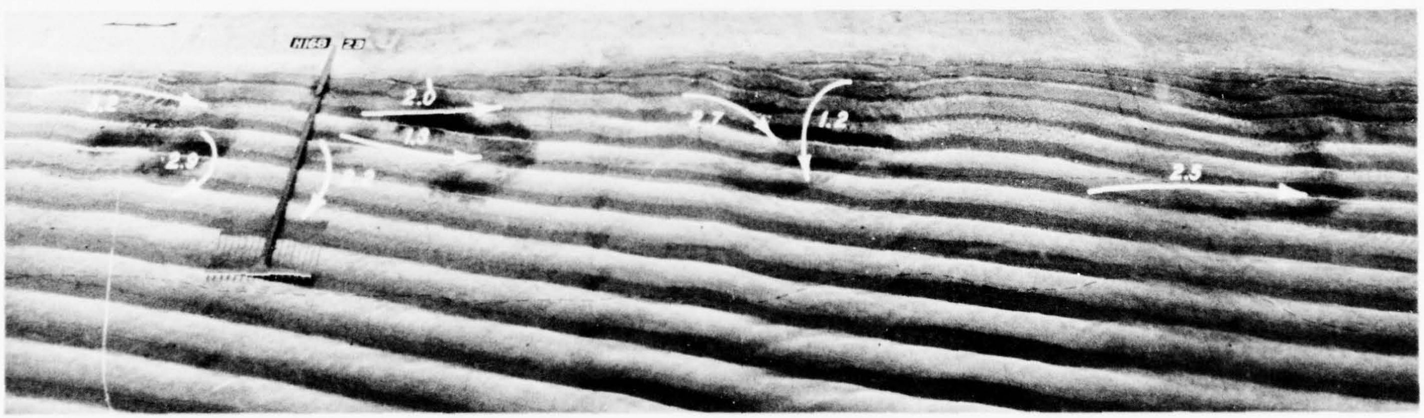


Photo 4. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 7-sec, 11-ft waves from west at mllw

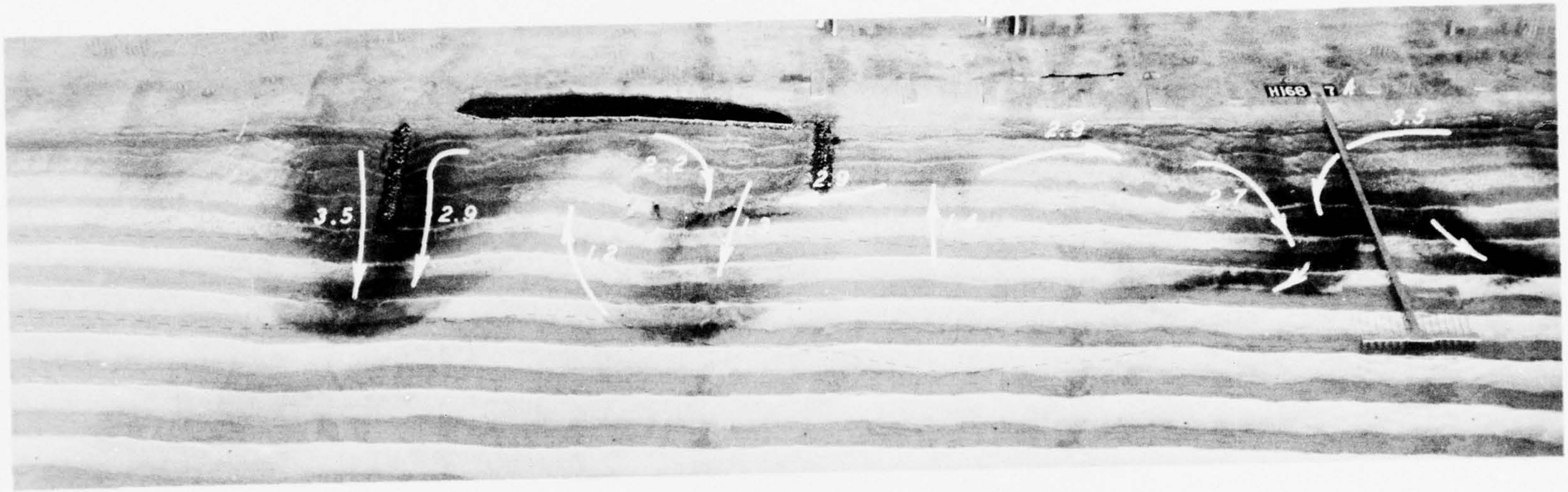


Photo 5. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 7-sec, 11-ft waves from west at mhhw

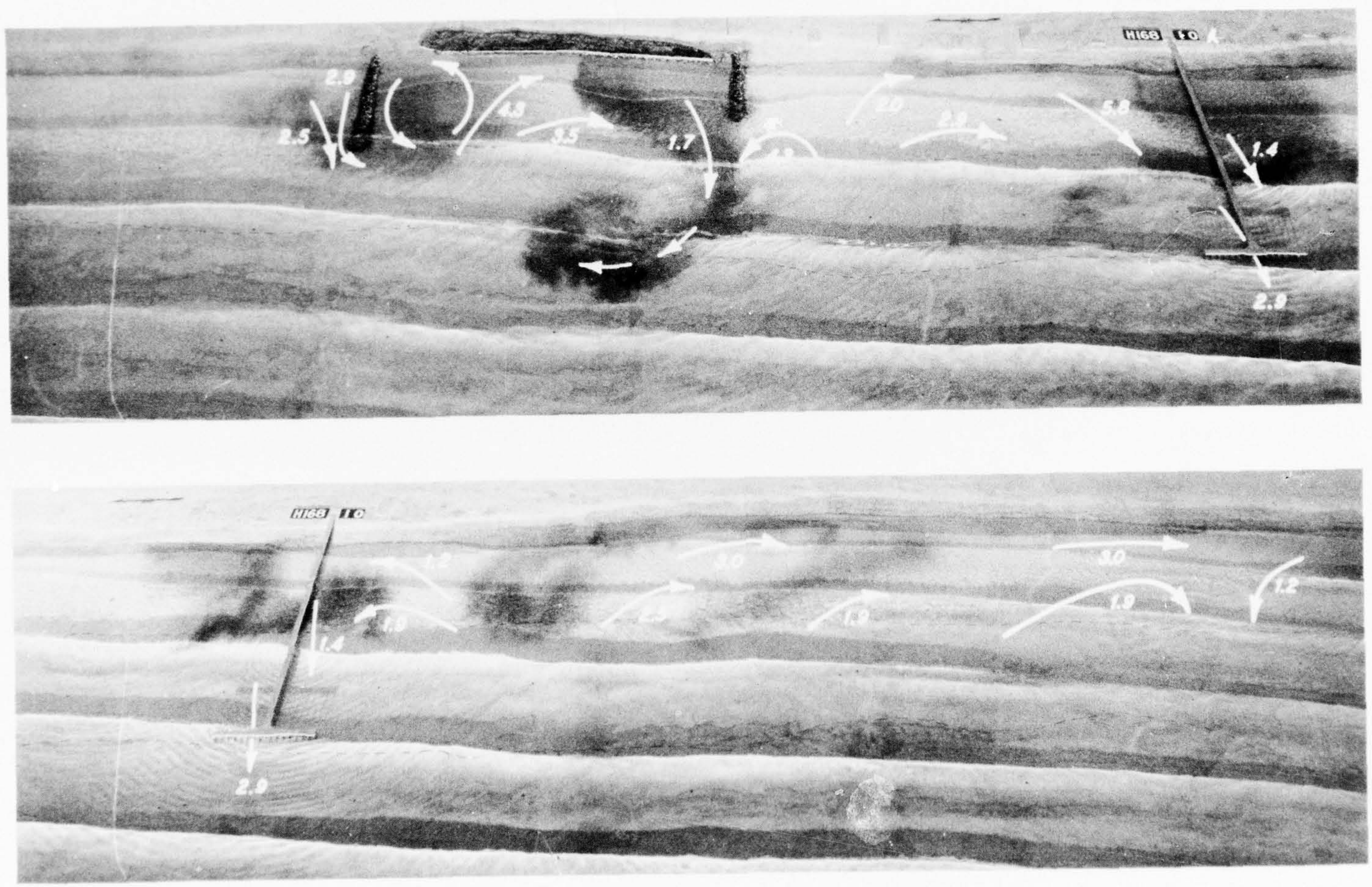


Photo 6. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 14-sec, 9-ft waves from west at mhhw

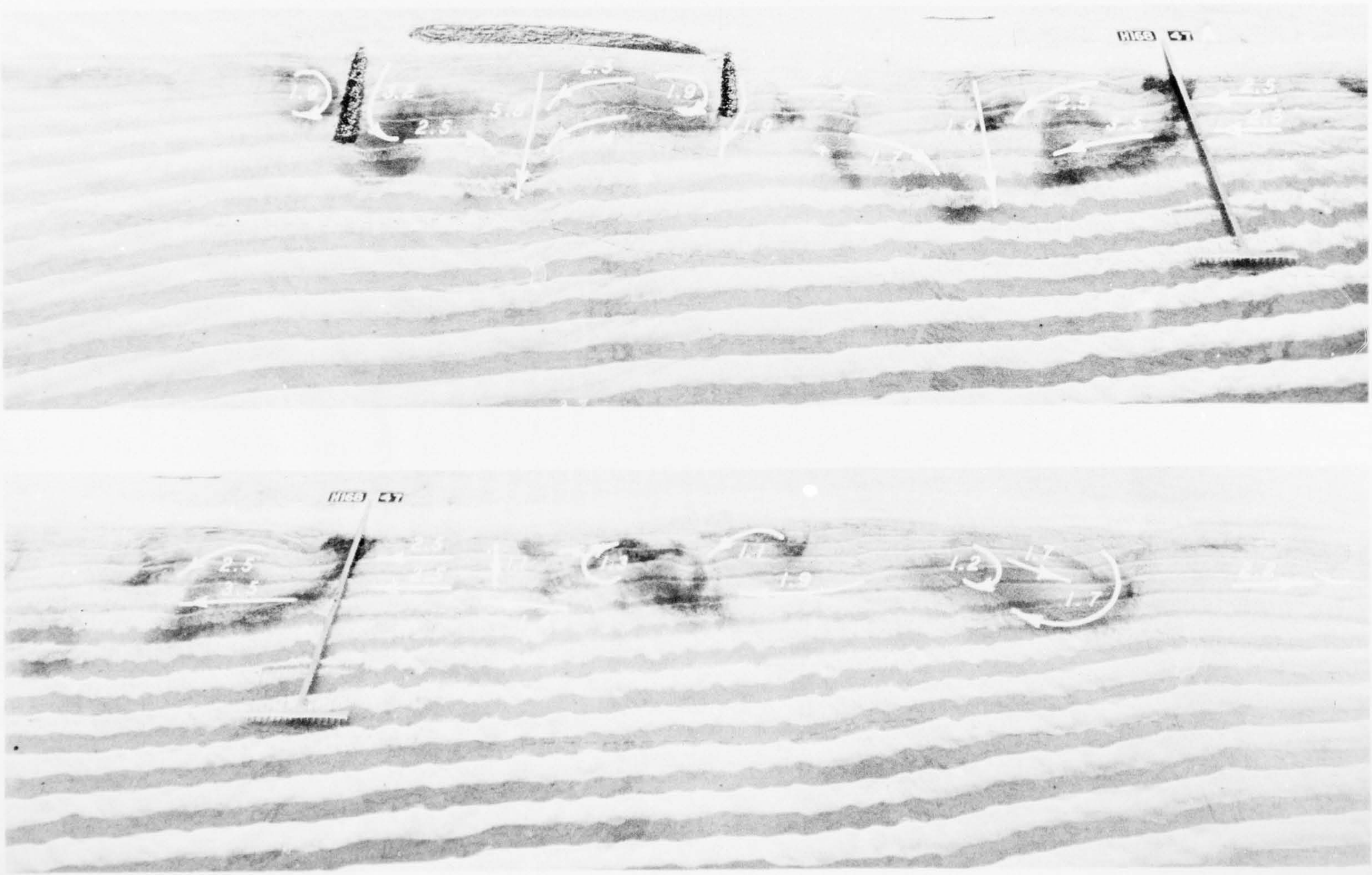


Photo 7. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 7-sec, 11-ft waves from S60°W at mllw

Photo 8. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 7-sec, 11-ft waves from S60°W at mhhw

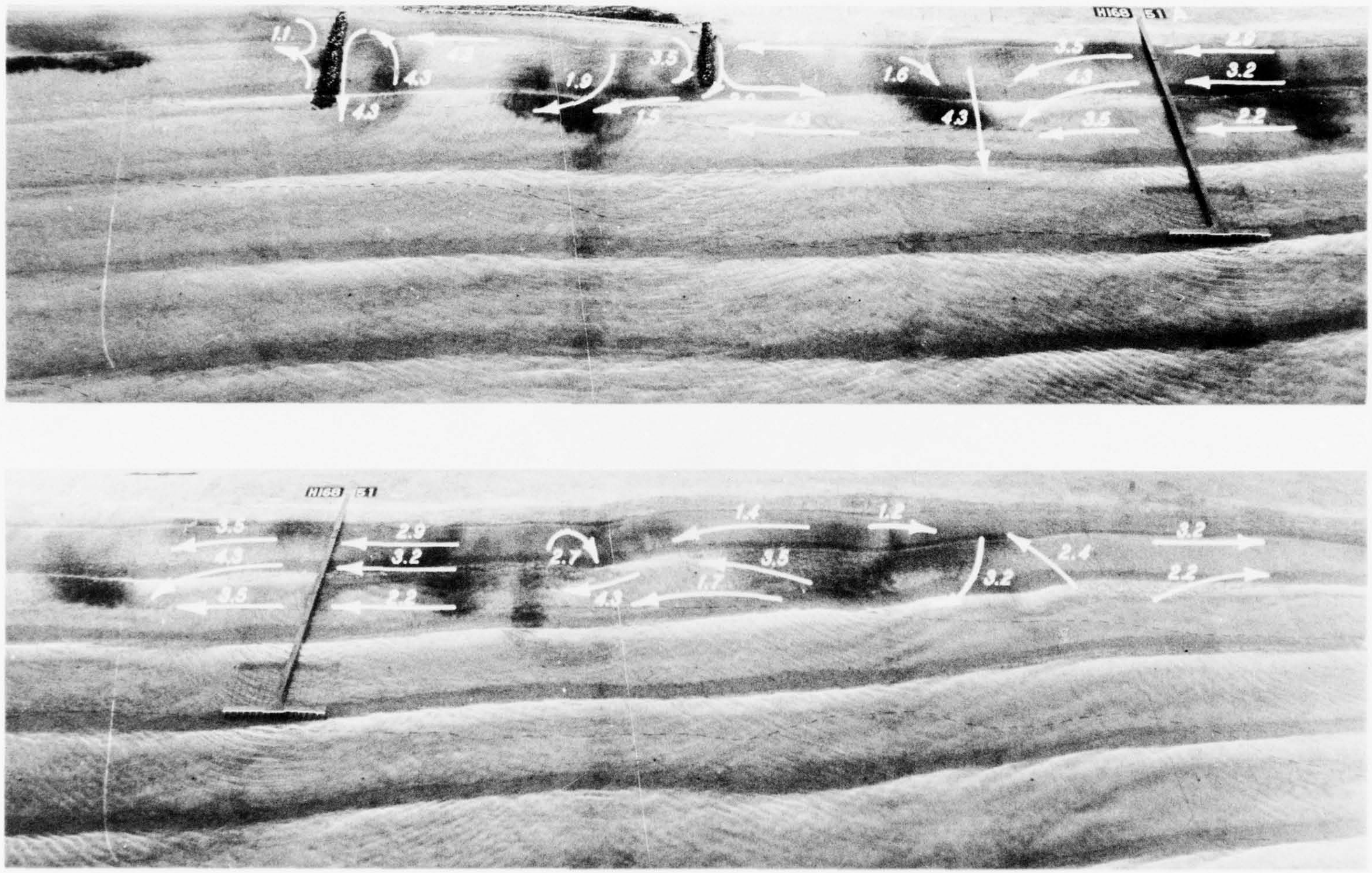


Photo 9. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions; 14-sec, 7-ft waves from S60°W at mhhw

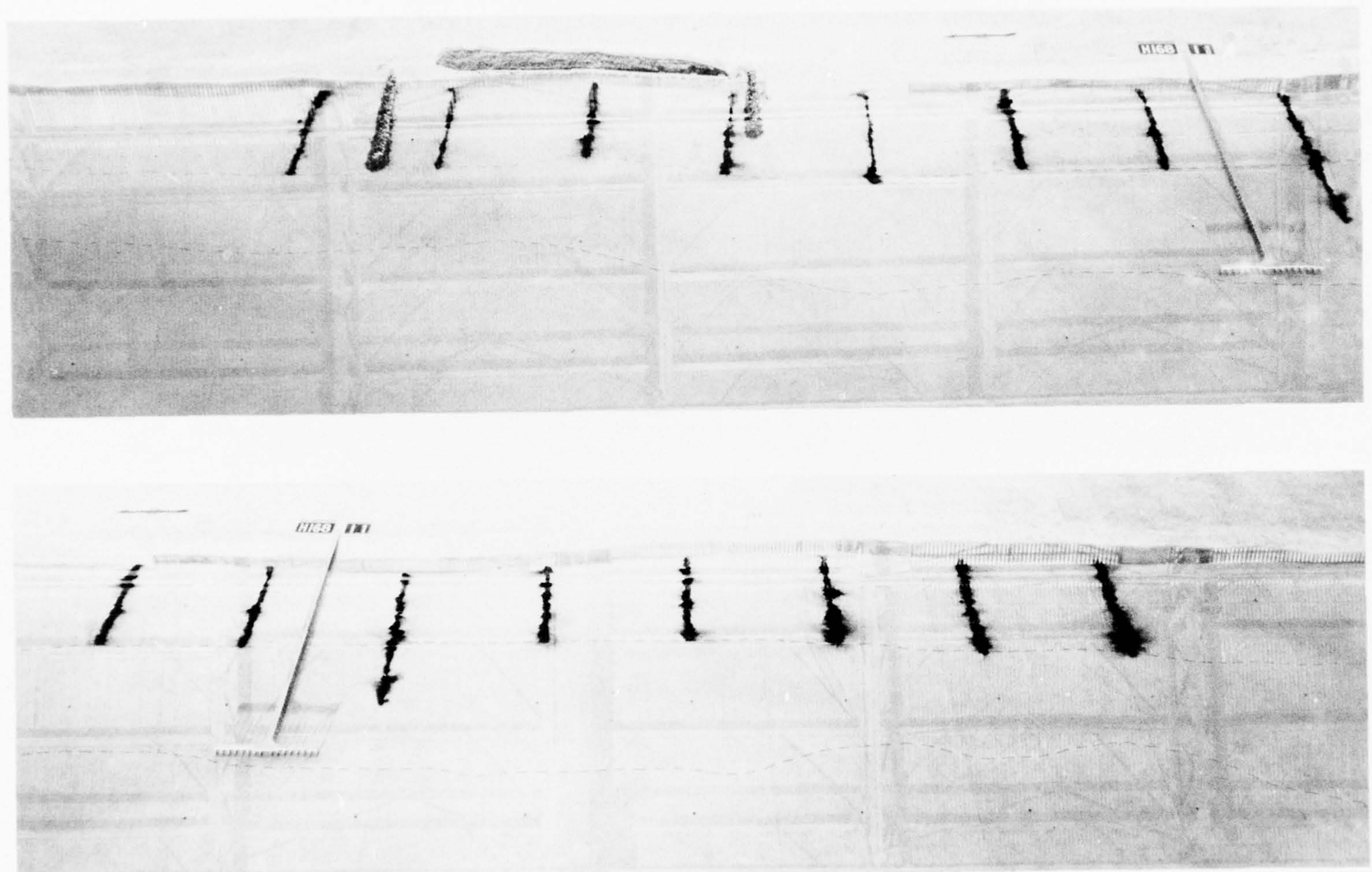


Photo 10. Tracer placement for existing conditions

Photo 11. Typical tracer movement for existing conditions resulting from 7-sec, 7-ft waves from N60°W at mllw

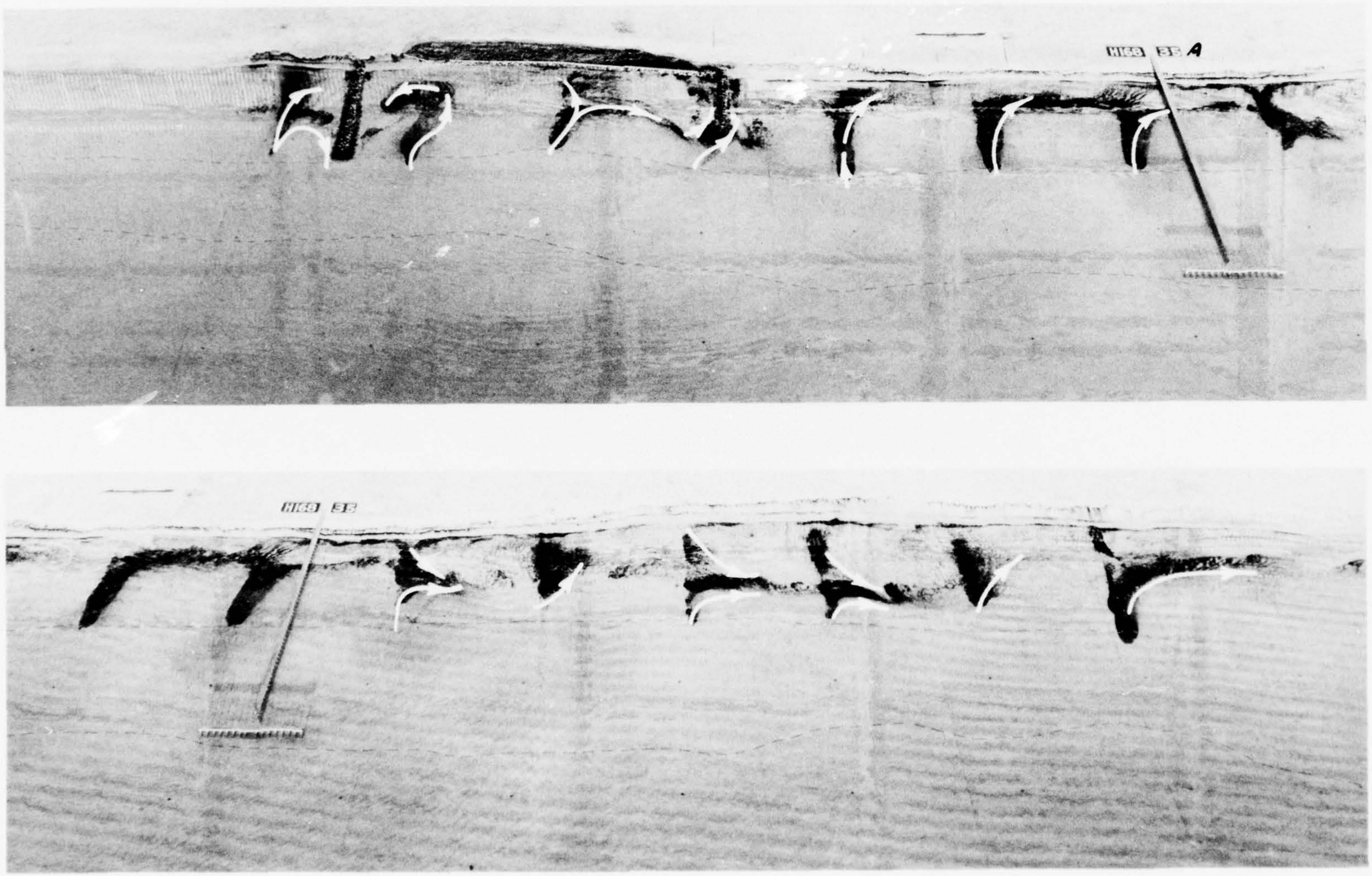


Photo 12. Typical tracer movement for existing conditions resulting from 7-sec, 7-ft waves from N60°W at mhhw

Photo 13. Typical tracer movement for existing conditions resulting from 7-sec, 11-ft waves from west at mllw

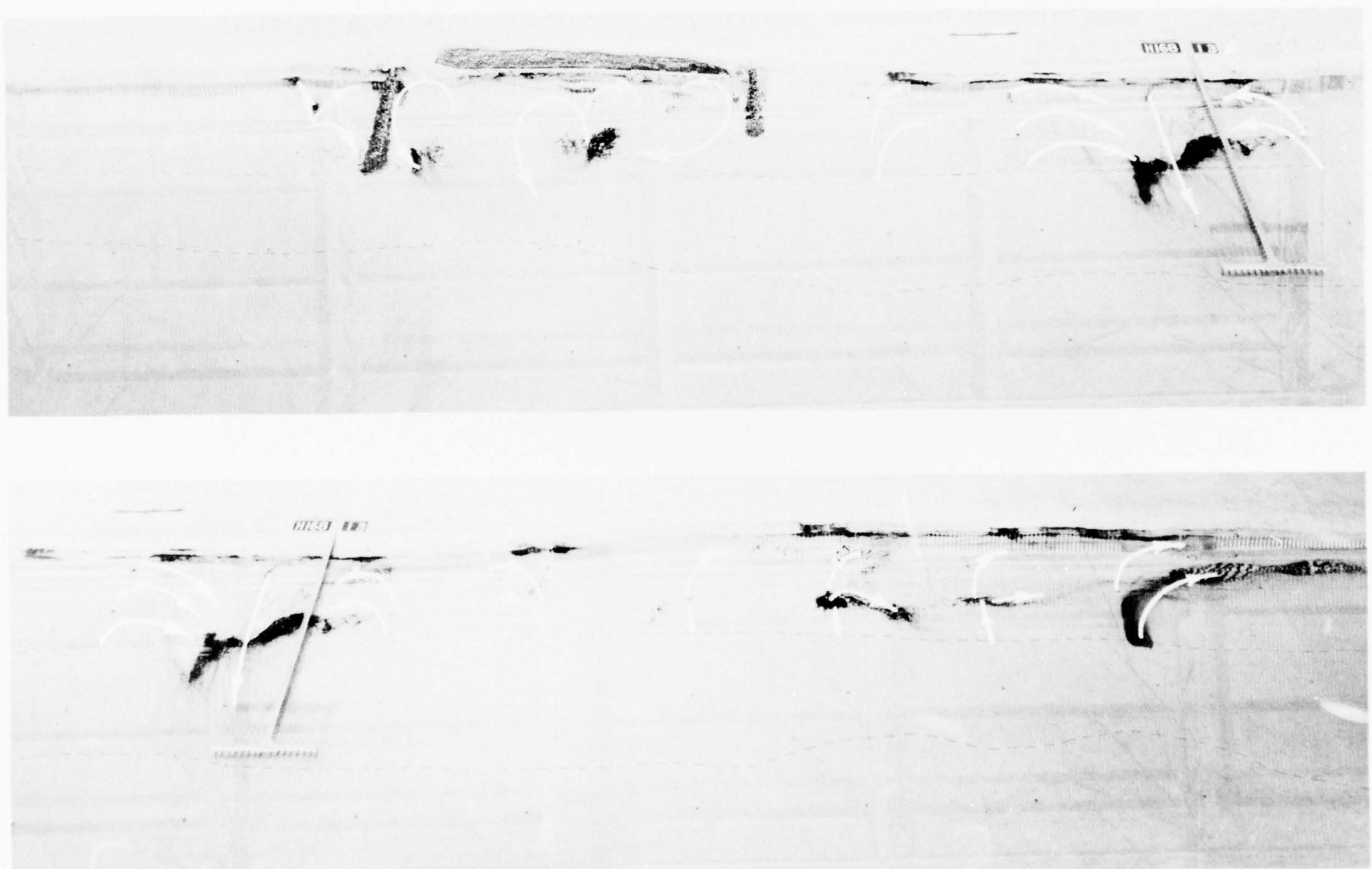


Photo 14. Typical tracer movement for existing conditions resulting from

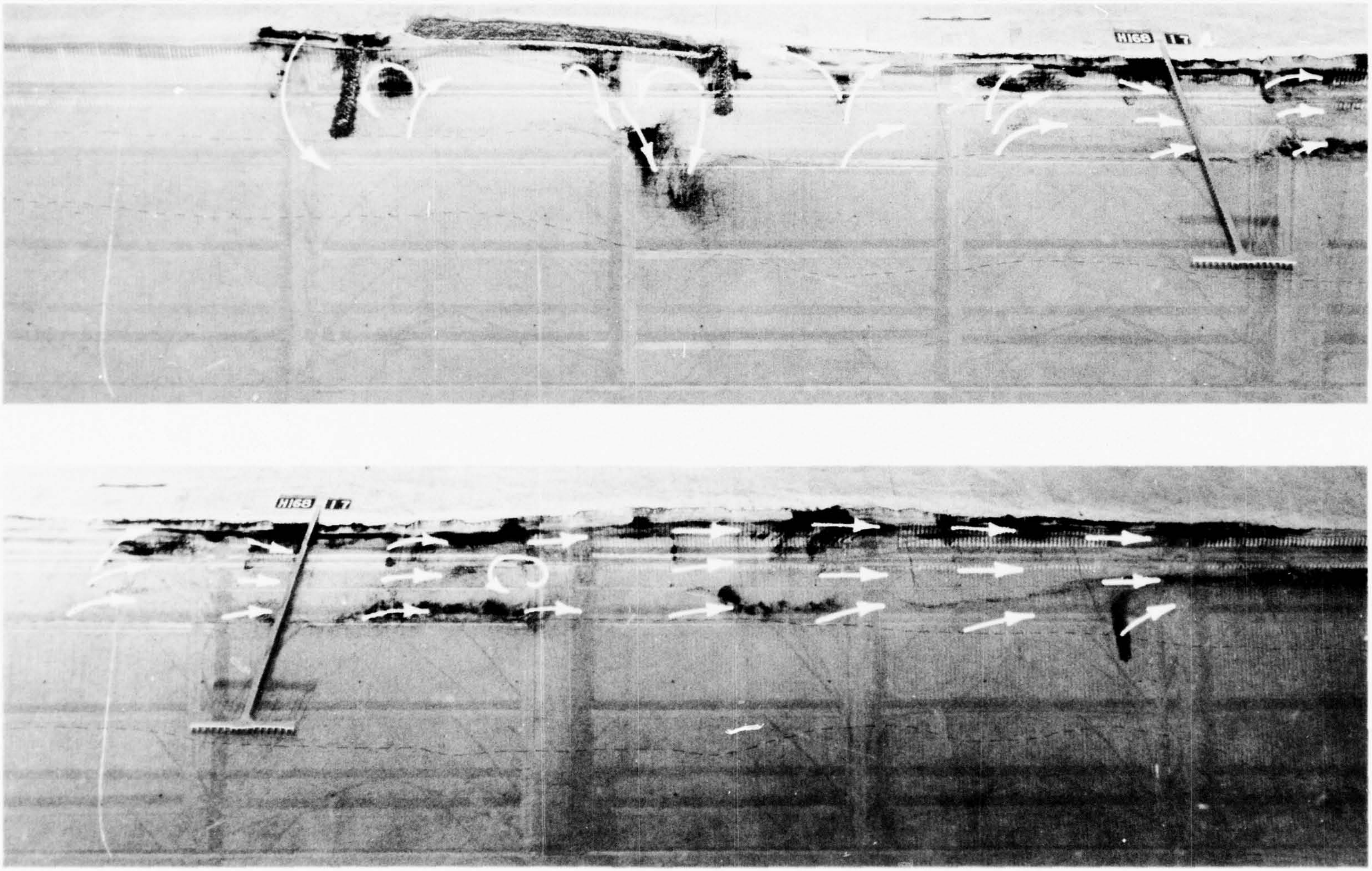


Photo 15. Typical tracer movement for existing conditions resulting from 14-sec, 9-ft waves from west at mhhw

Photo 16. Typical tracer movement for existing conditions resulting from 7-sec, 11-ft waves from S60°W at mllw

Photo 17. Typical tracer movement for existing conditions resulting from 7-sec, 11-ft waves from S60°W at mhhw

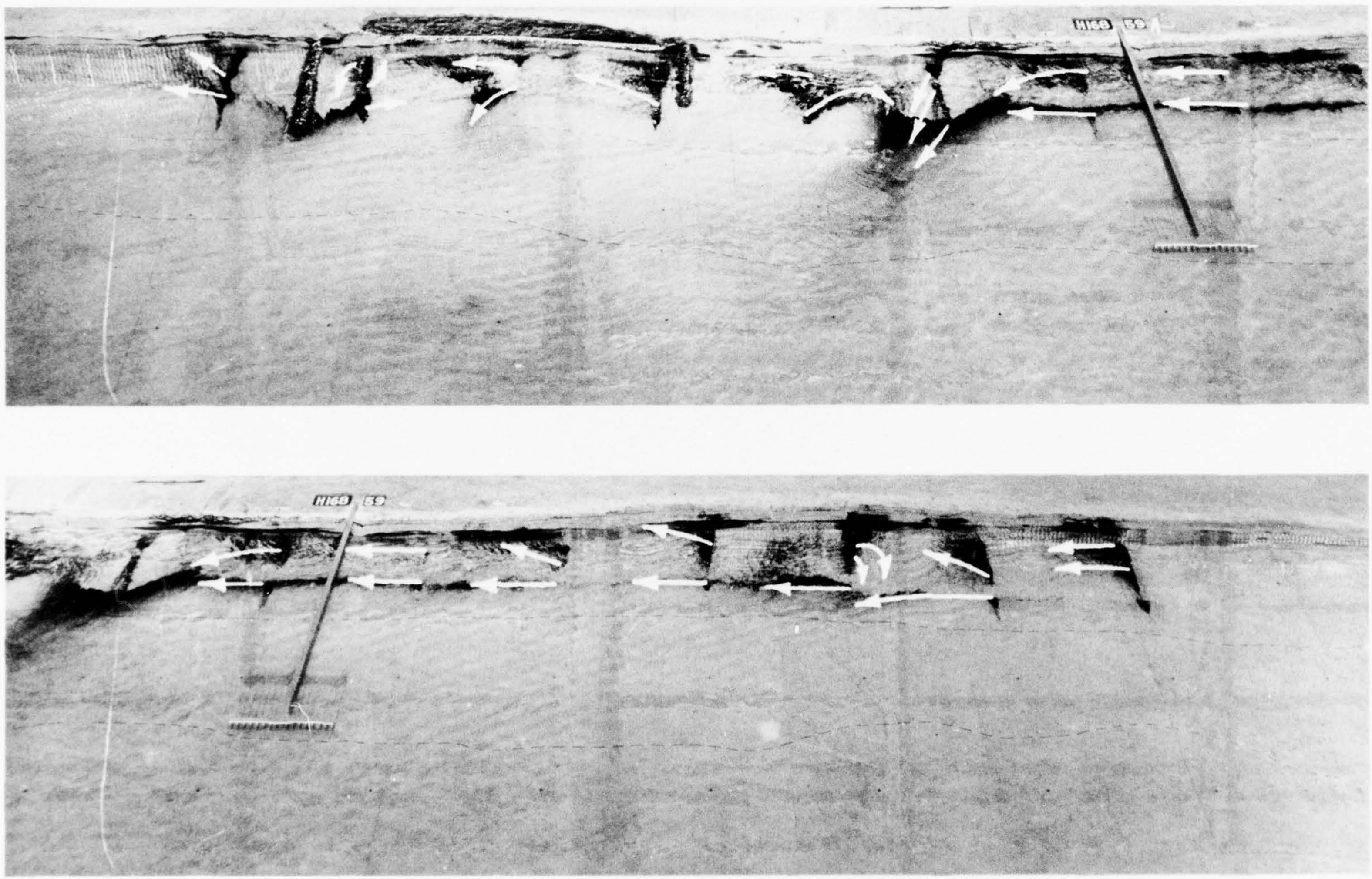


Photo 18. Typical tracer movement for existing conditions resulting from 14-sec, 7-ft waves from S60°W at mhhw

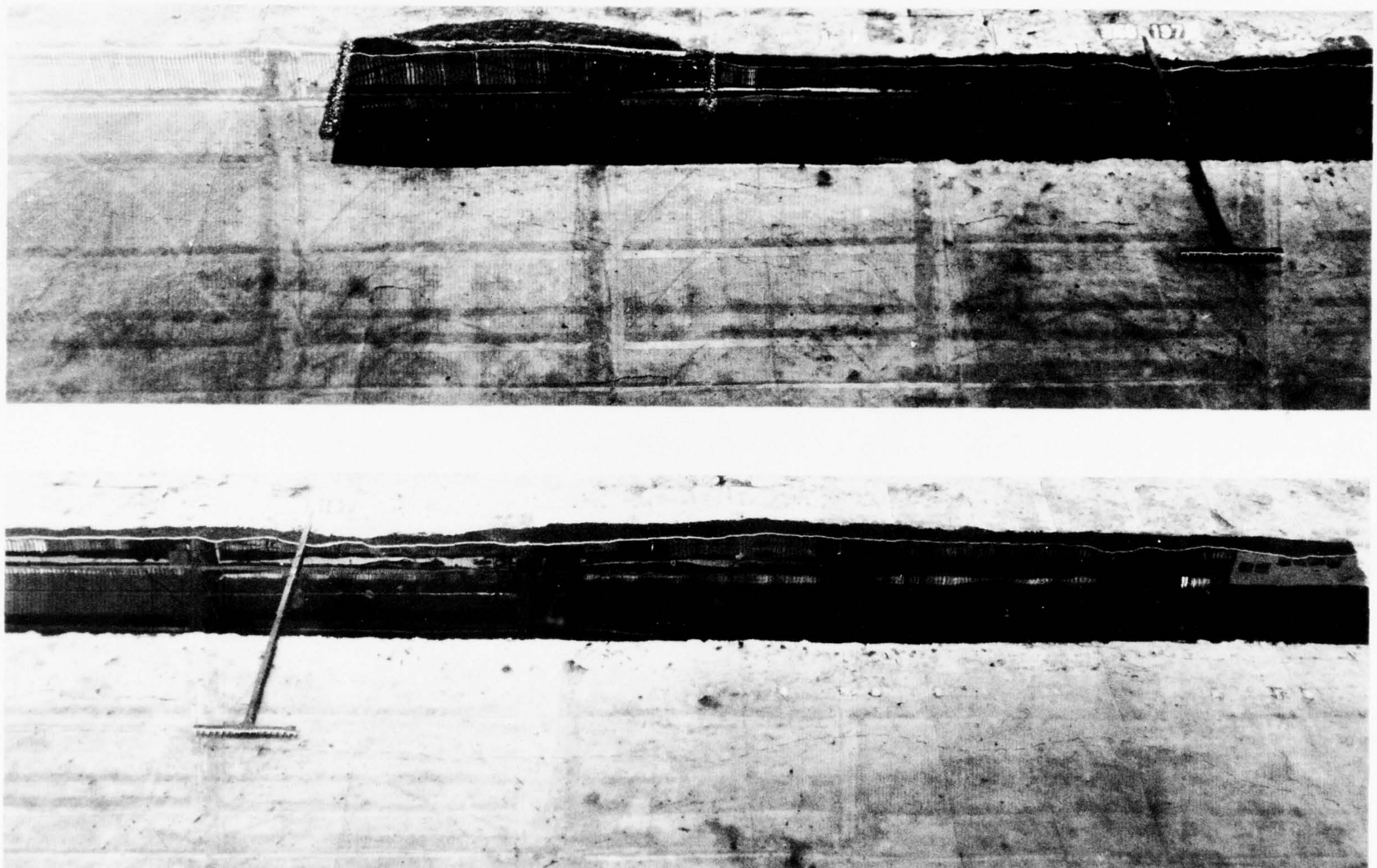
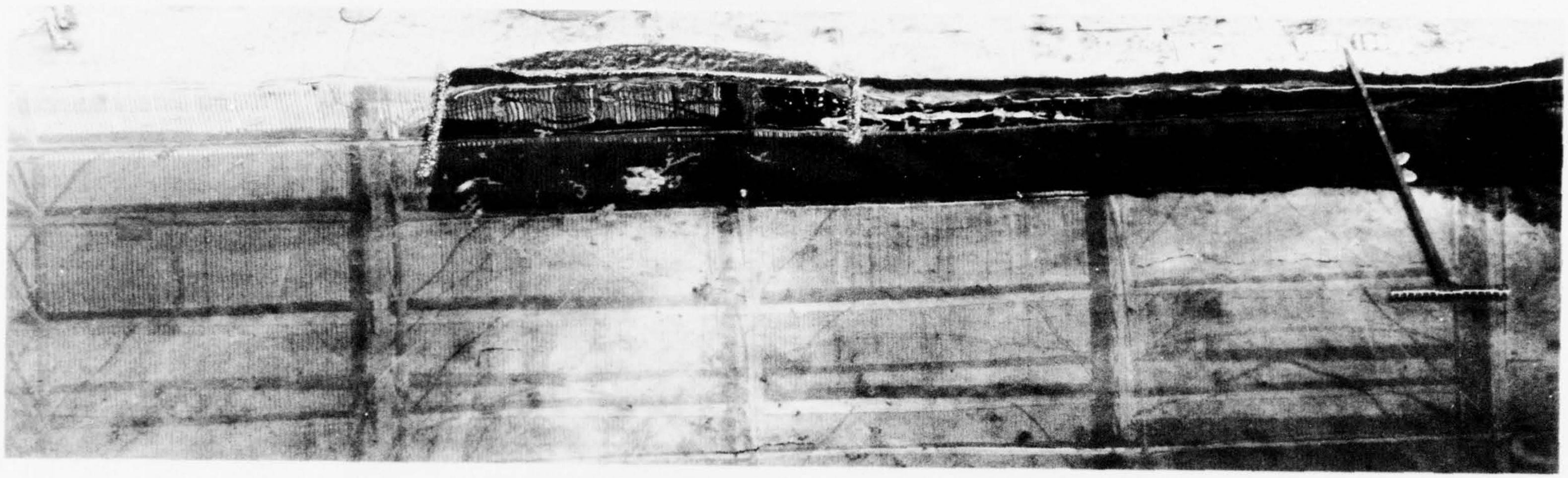


Photo 19. Semimovable-bed tracer placement for existing conditions



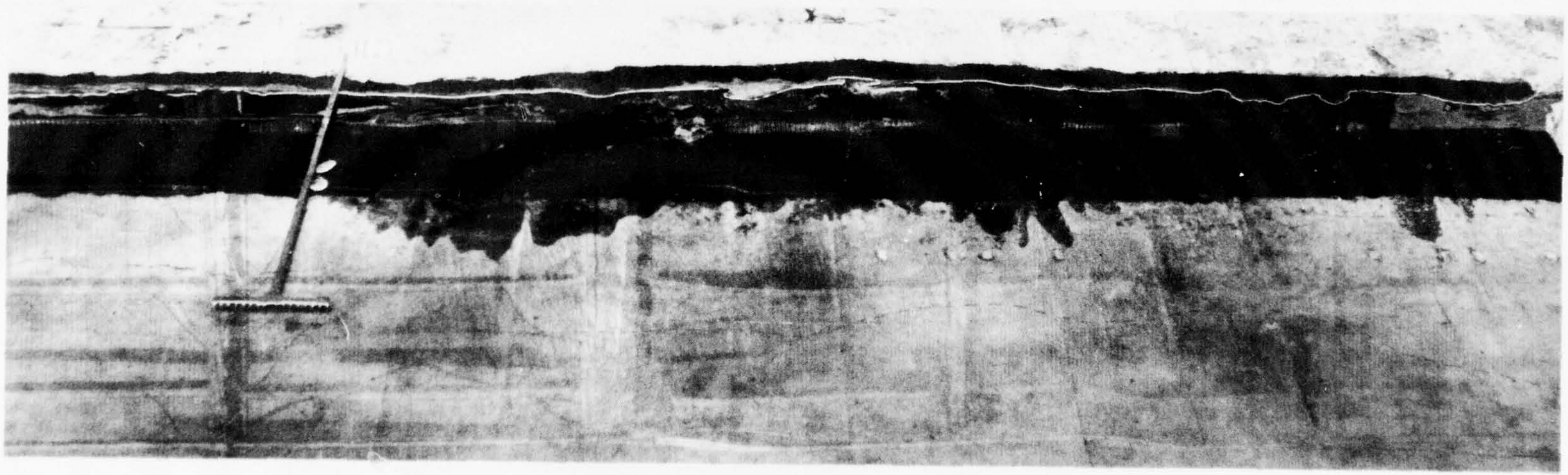


Photo 20. Semimovable-bed tracer deposits for existing conditions resulting from 7-sec, 4-ft waves from S60°W at mhhw

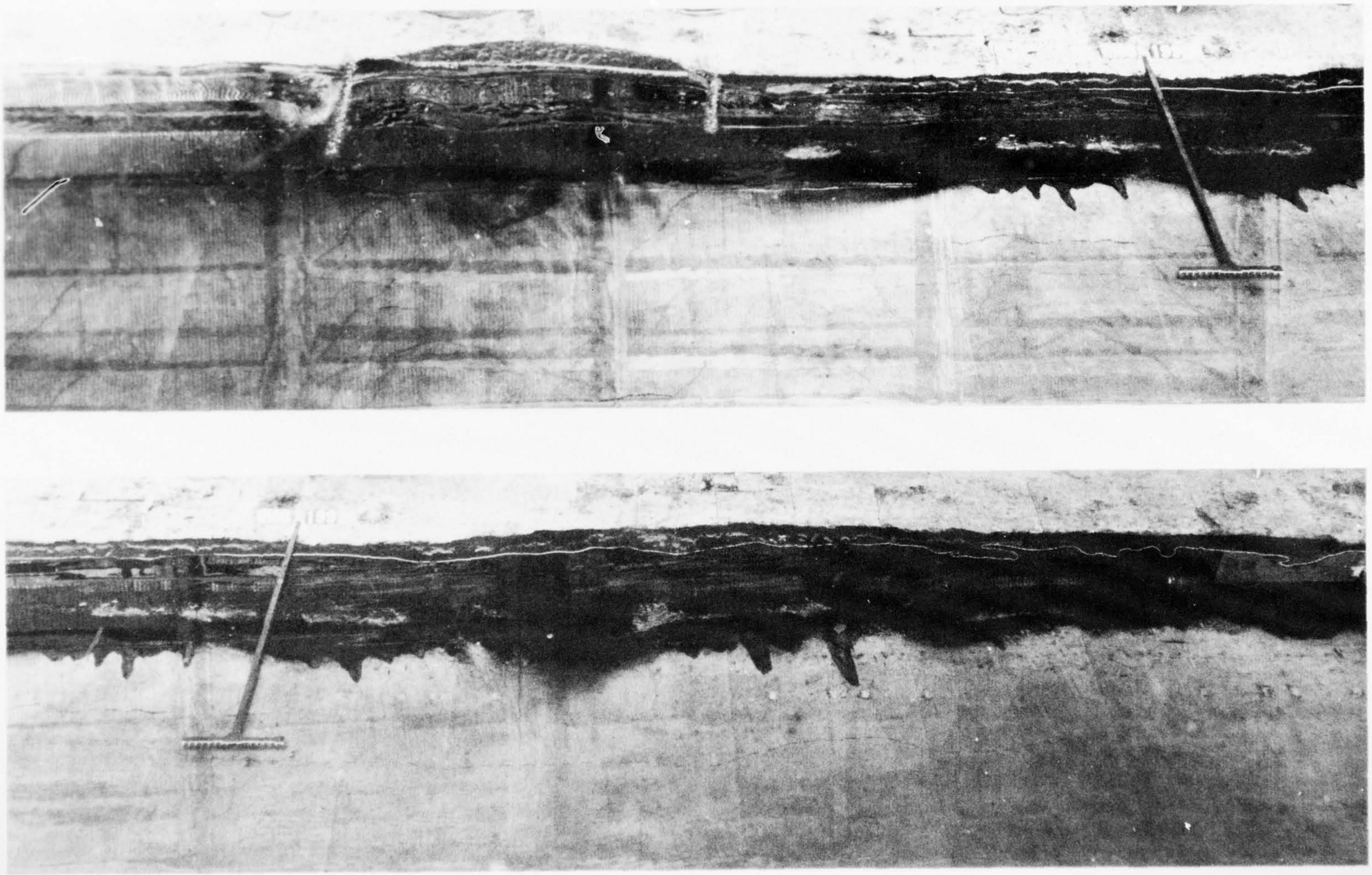


Photo 21. Semimovable-bed tracer deposits for existing conditions resulting from 7-sec, 11-ft waves from S60°W at mhhw

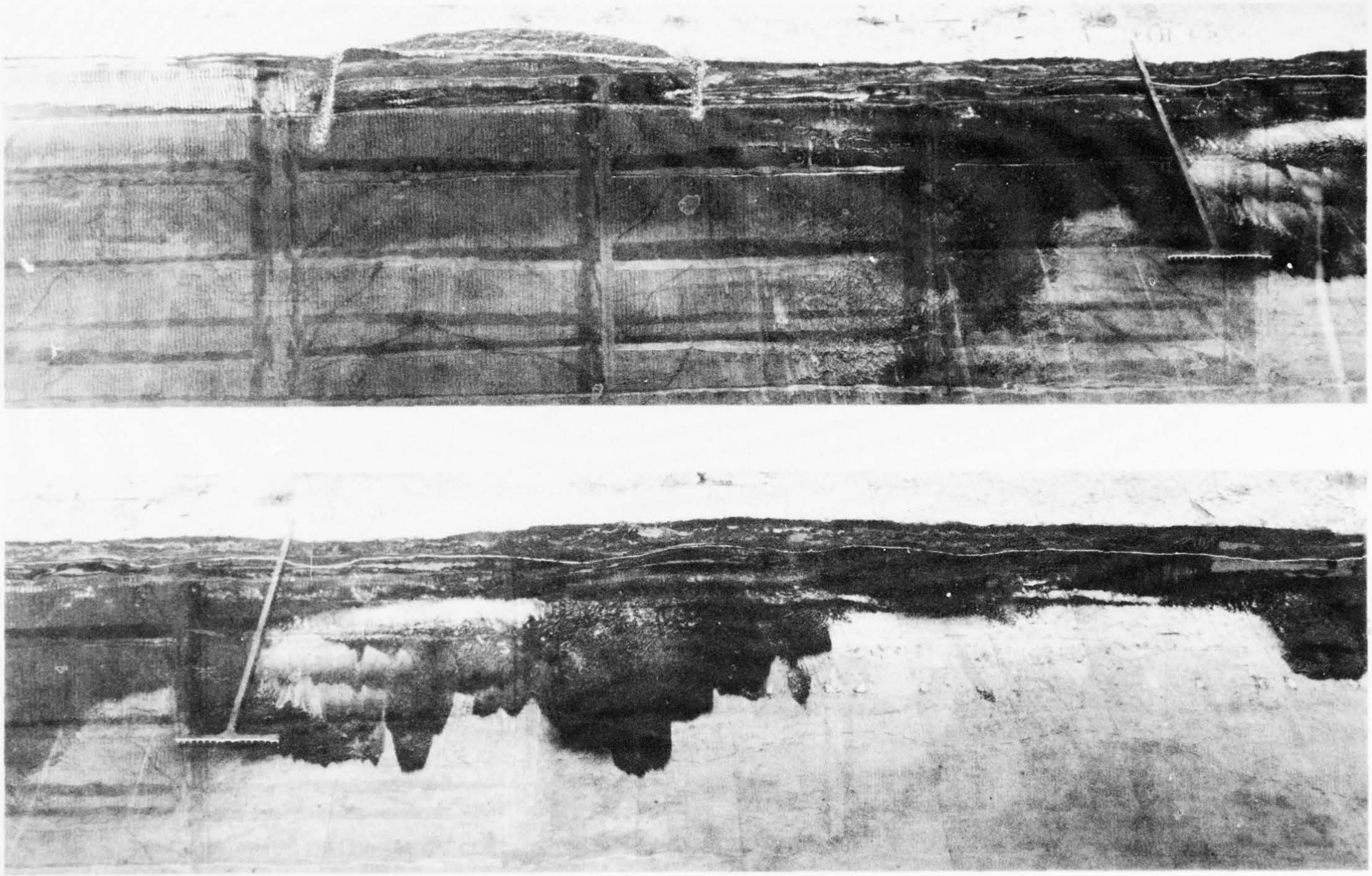


Photo 22. Semimovable-bed tracer deposits for existing conditions resulting from 14-sec, 7-ft waves from S60°W at mhhw

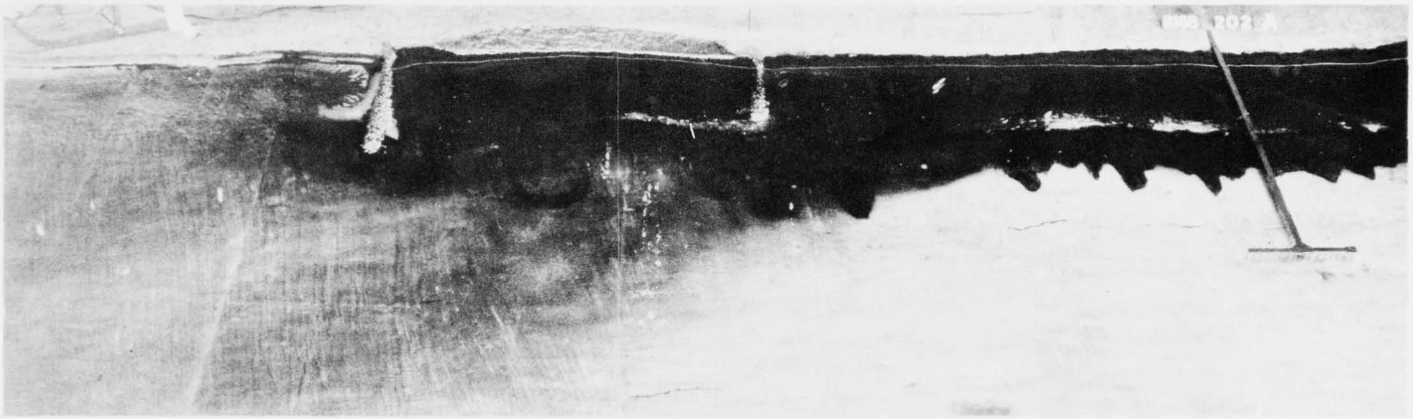


Photo 23. Semimovable-bed tracer deposits for existing conditions resulting from 7-sec, 11-ft waves from S60°W at mhhw

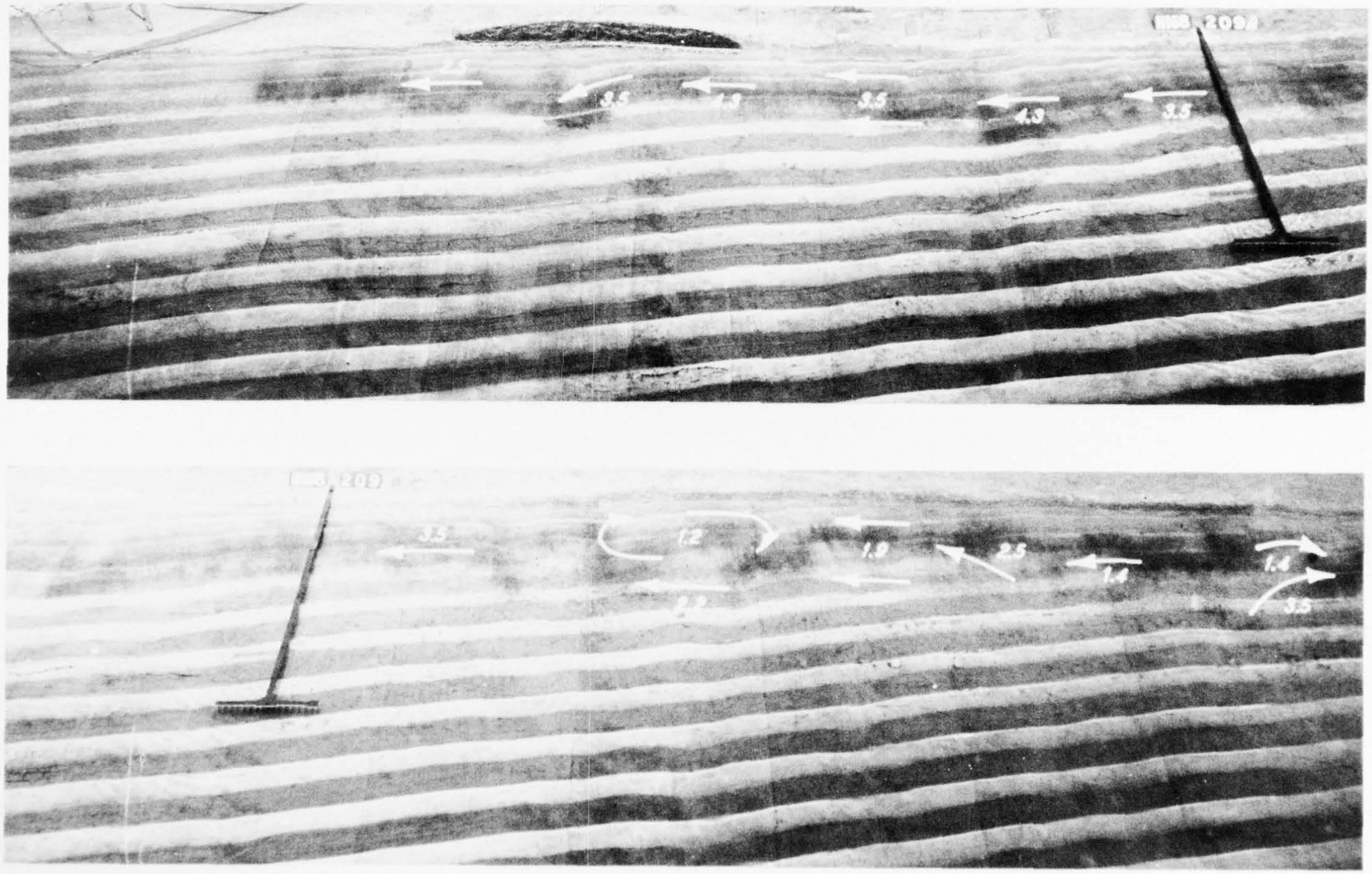


Photo 24. Typical wave and current patterns and current magnitudes (prototype feet per second) for existing conditions (groins removed); 7-sec, 11-ft waves from S60°W at mhhw

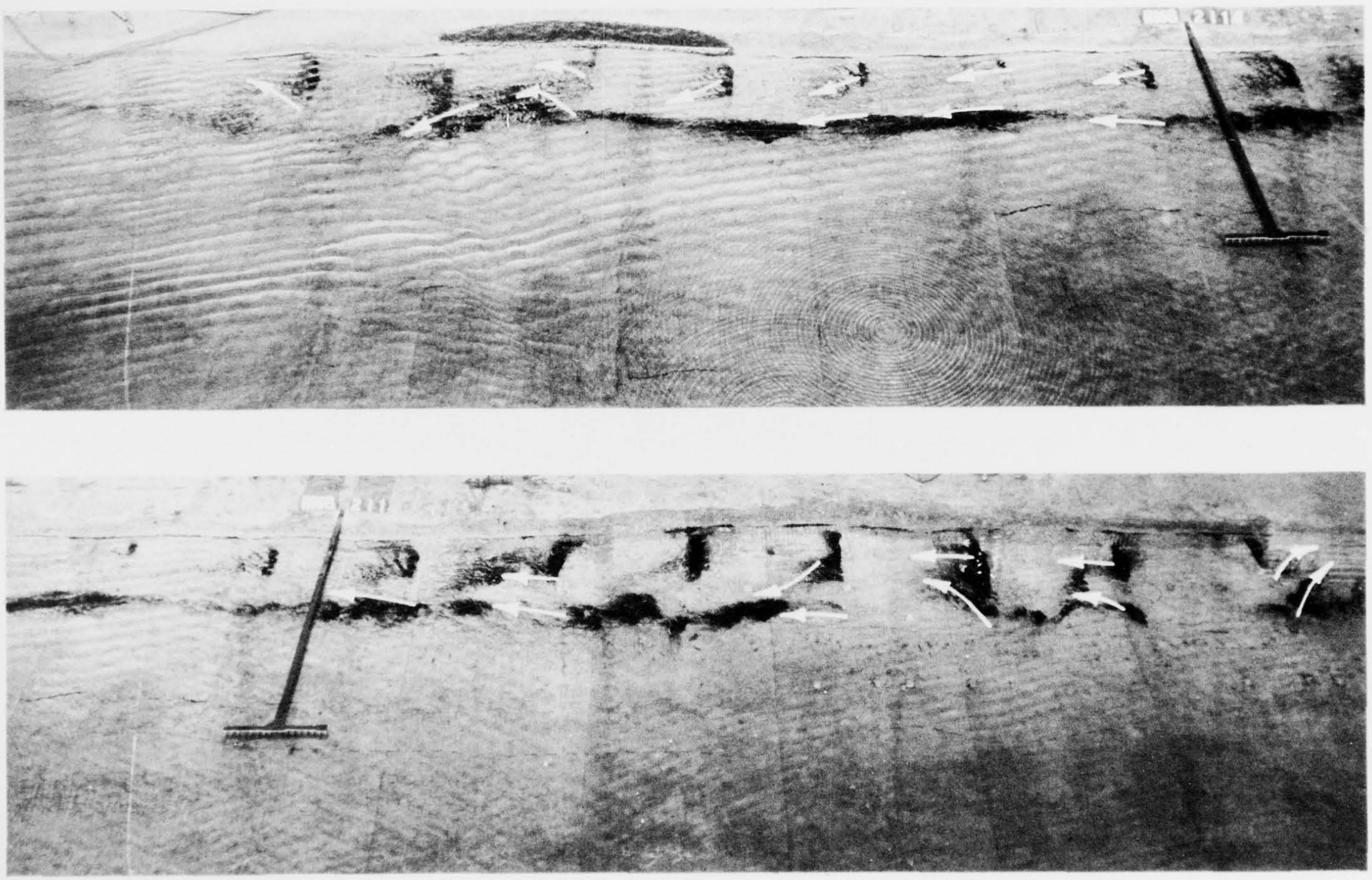


Photo 25. Typical tracer movement for existing conditions (groins removed) resulting from 7-sec, 11-ft waves from S60°W at mhhw

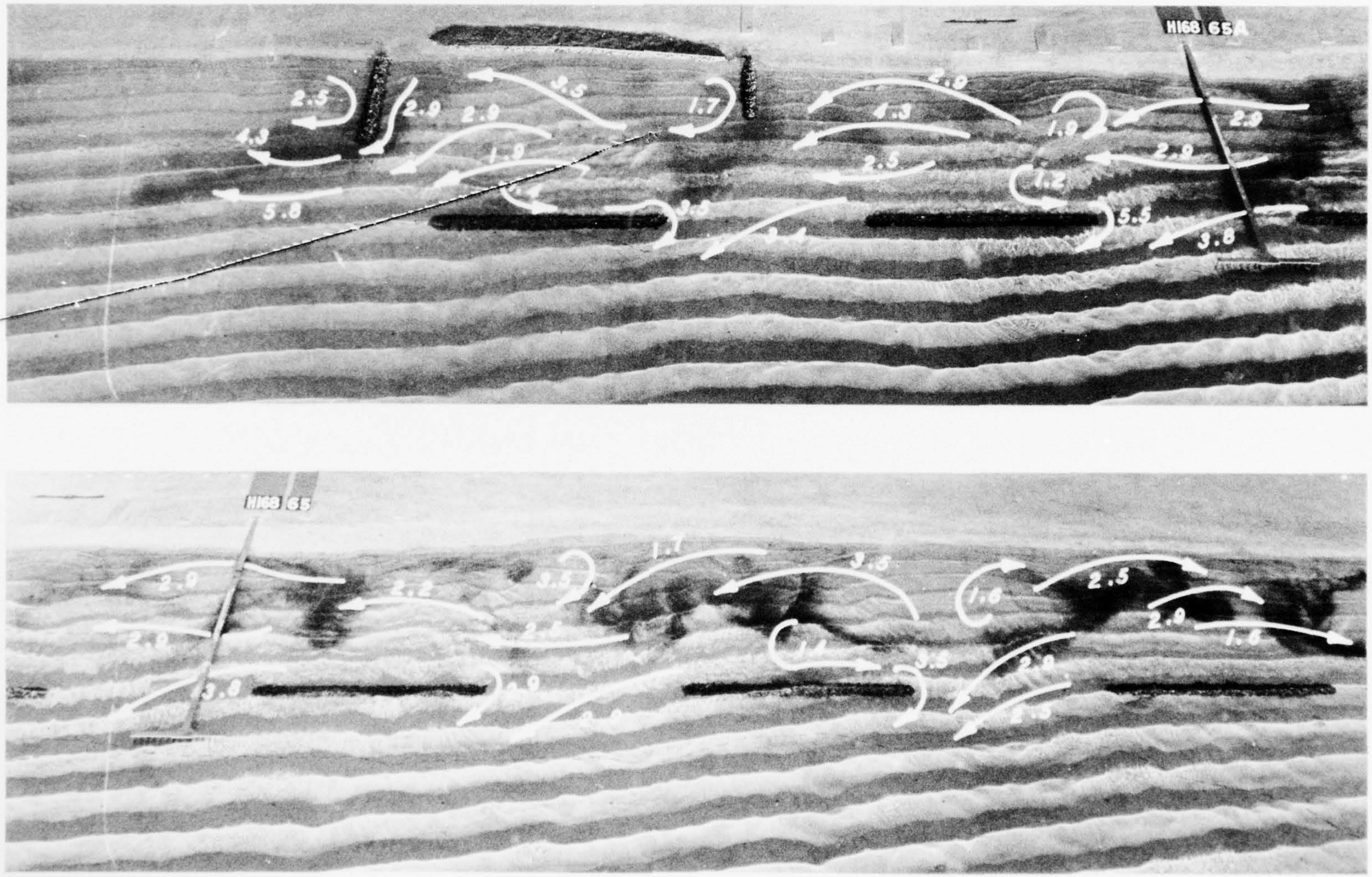


Photo 26. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 1; 7-sec, 11-ft waves from S60°W at mllw

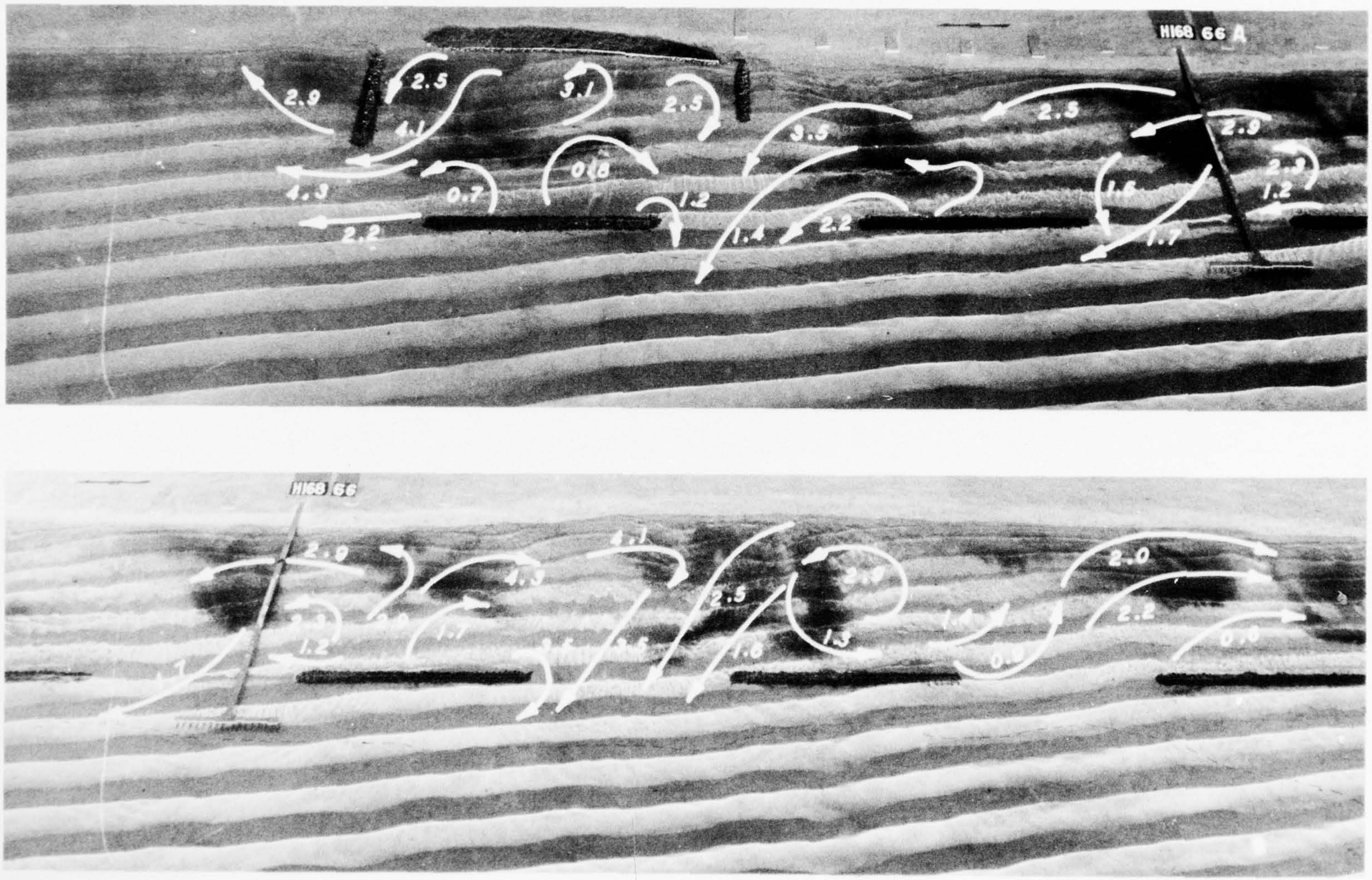


Photo 27. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 1; 7-sec, 11-ft waves from S60°W at mhhw

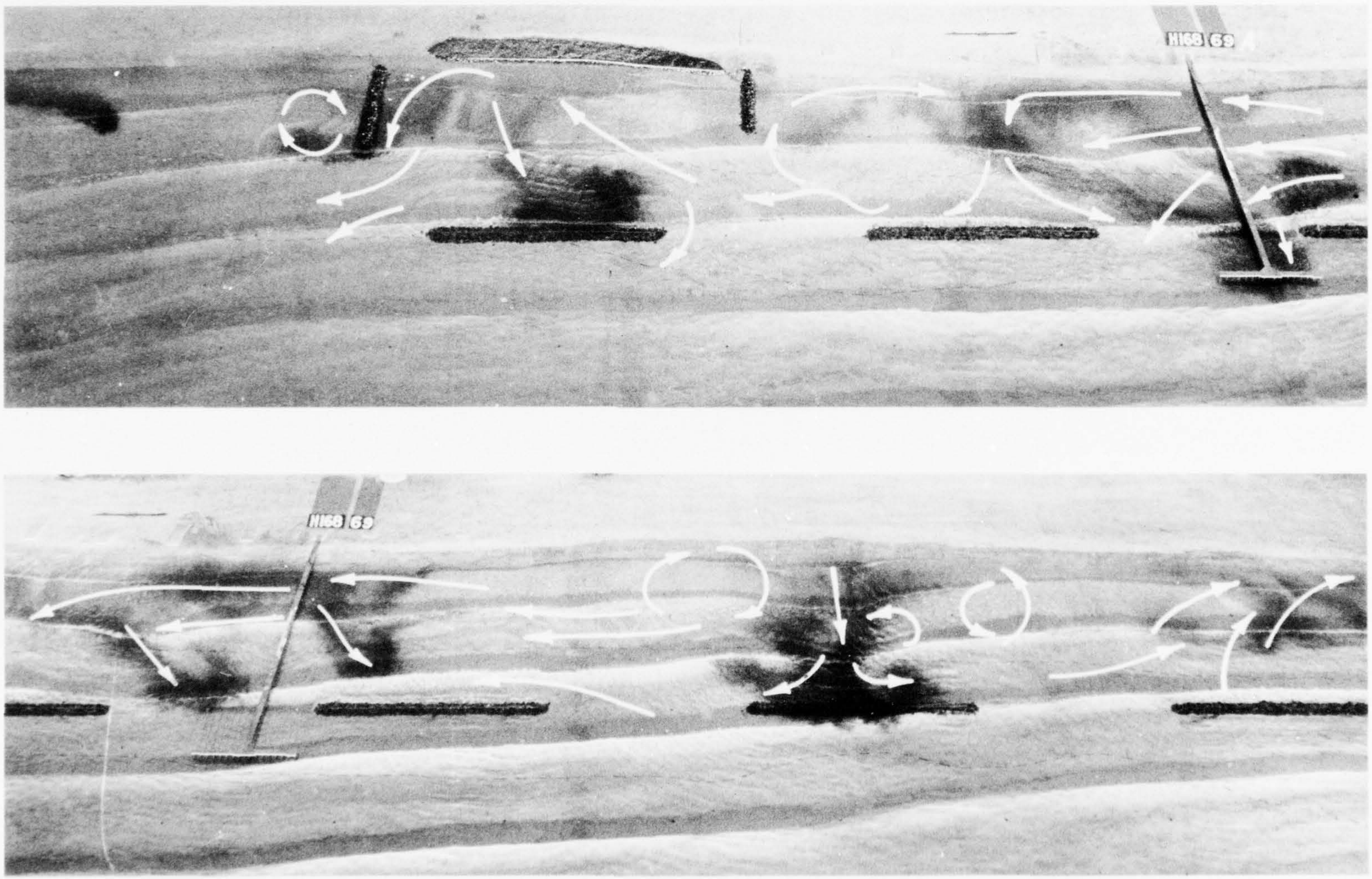


Photo 28. Typical wave and current patterns and current magnitudes (prototype feet per second)

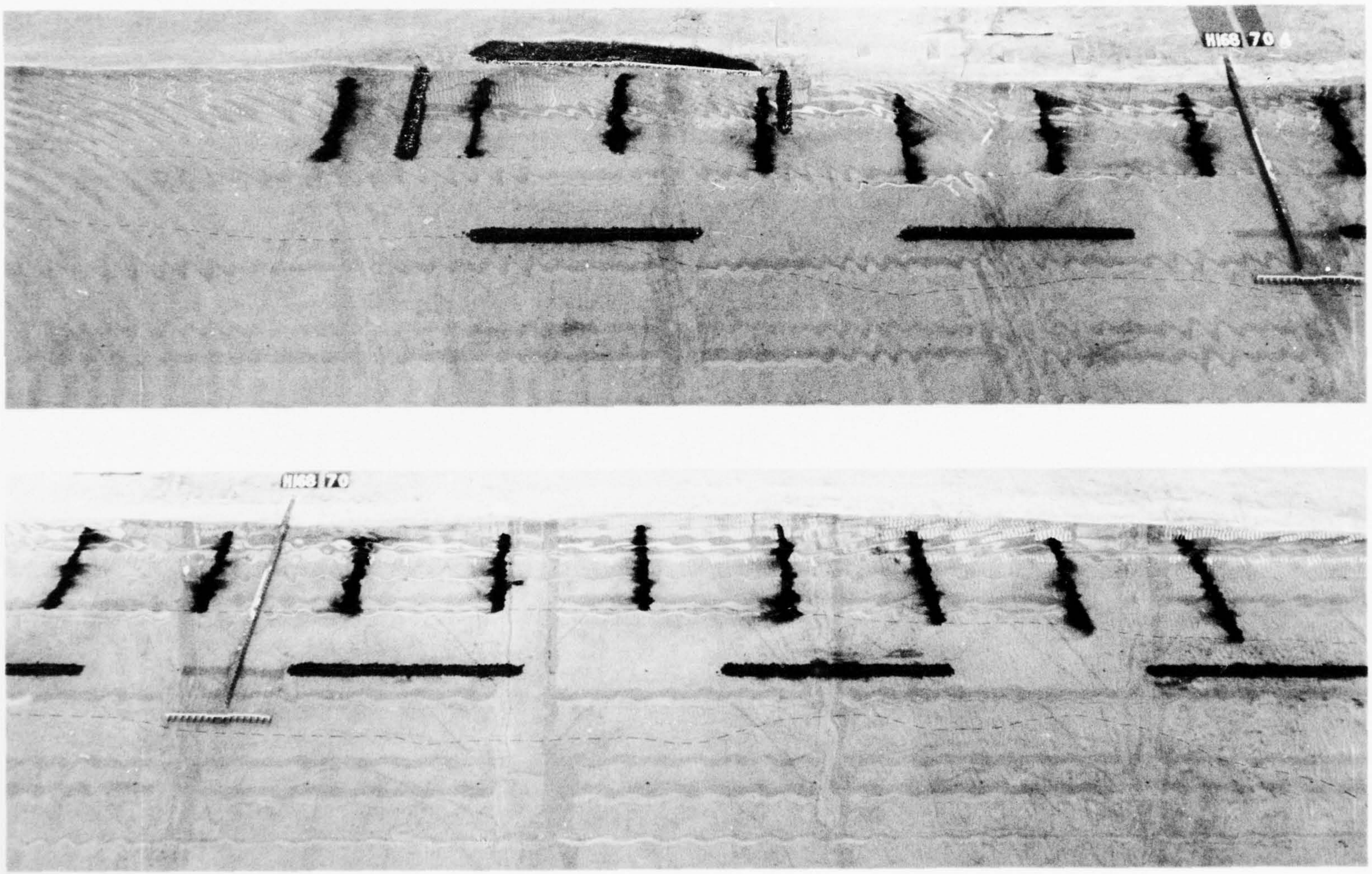


Photo 29. Typical tracer placement for Plan 1 before testing

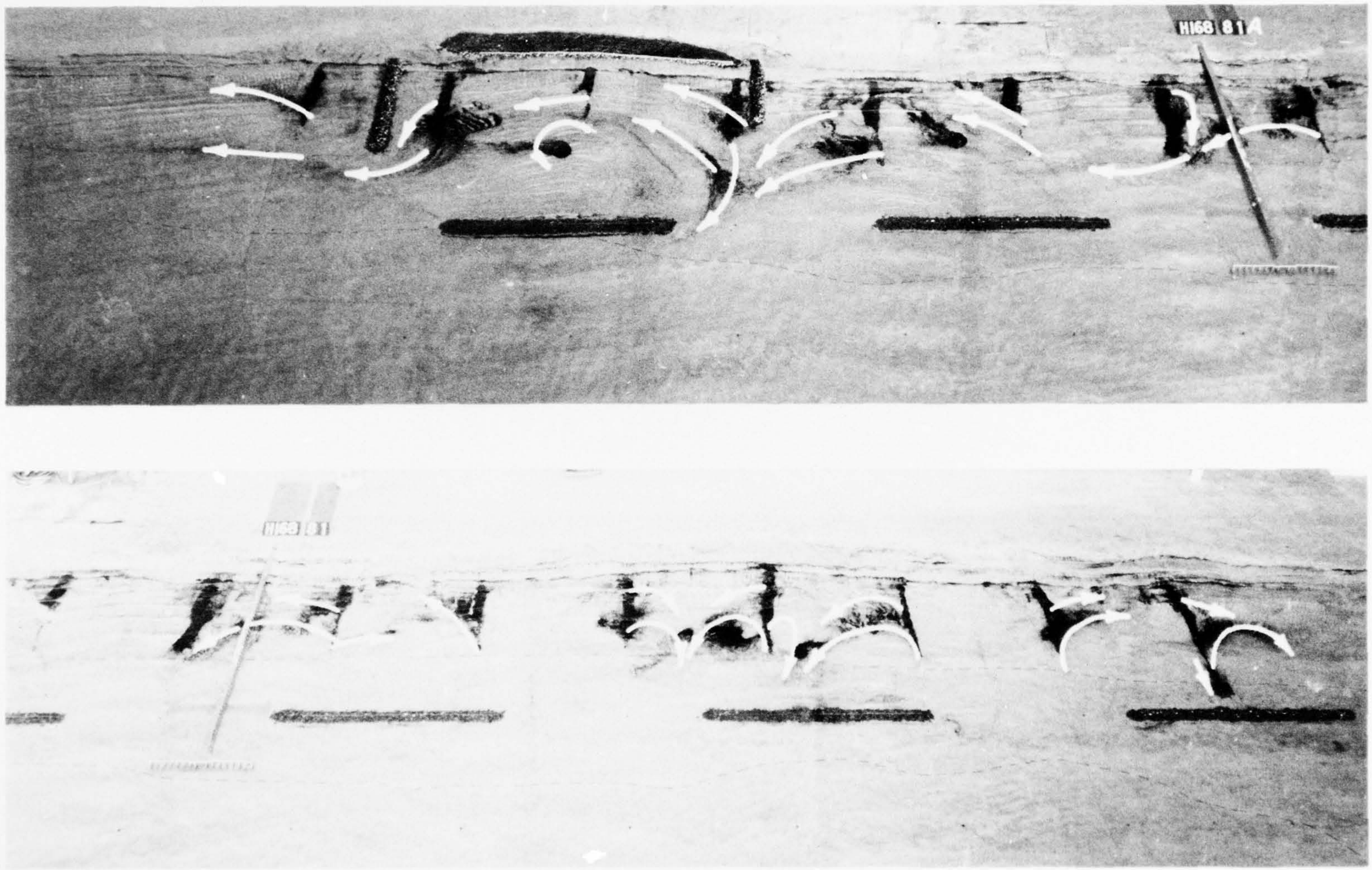


Photo 30. Typical tracer movement for Plan 1 resulting from 7-sec, 11-ft waves from S60°W at mllw

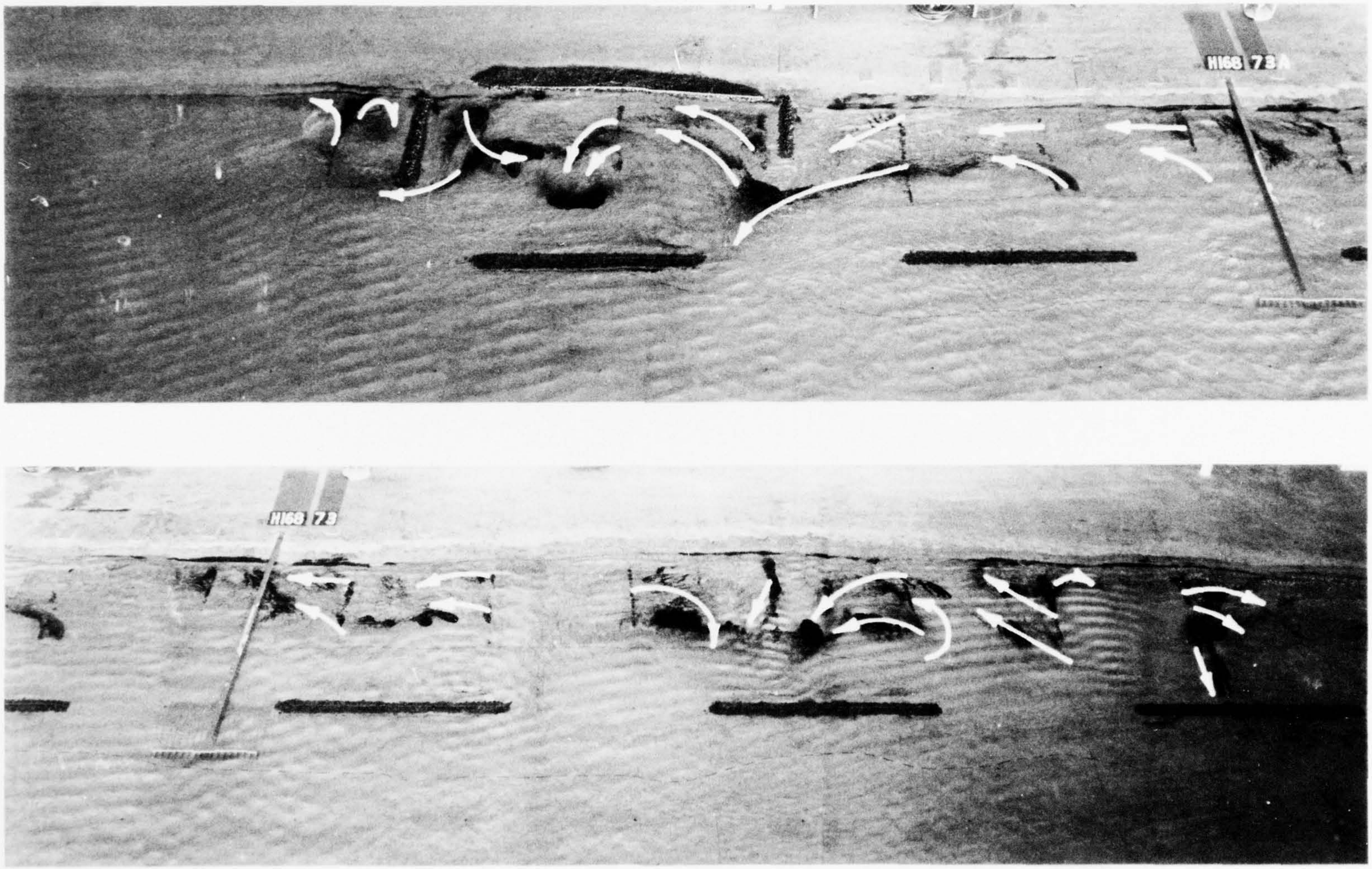


Photo 31. Typical tracer movement for Plan 1 resulting from 7-sec, 11-ft waves from S60°W at mhhw

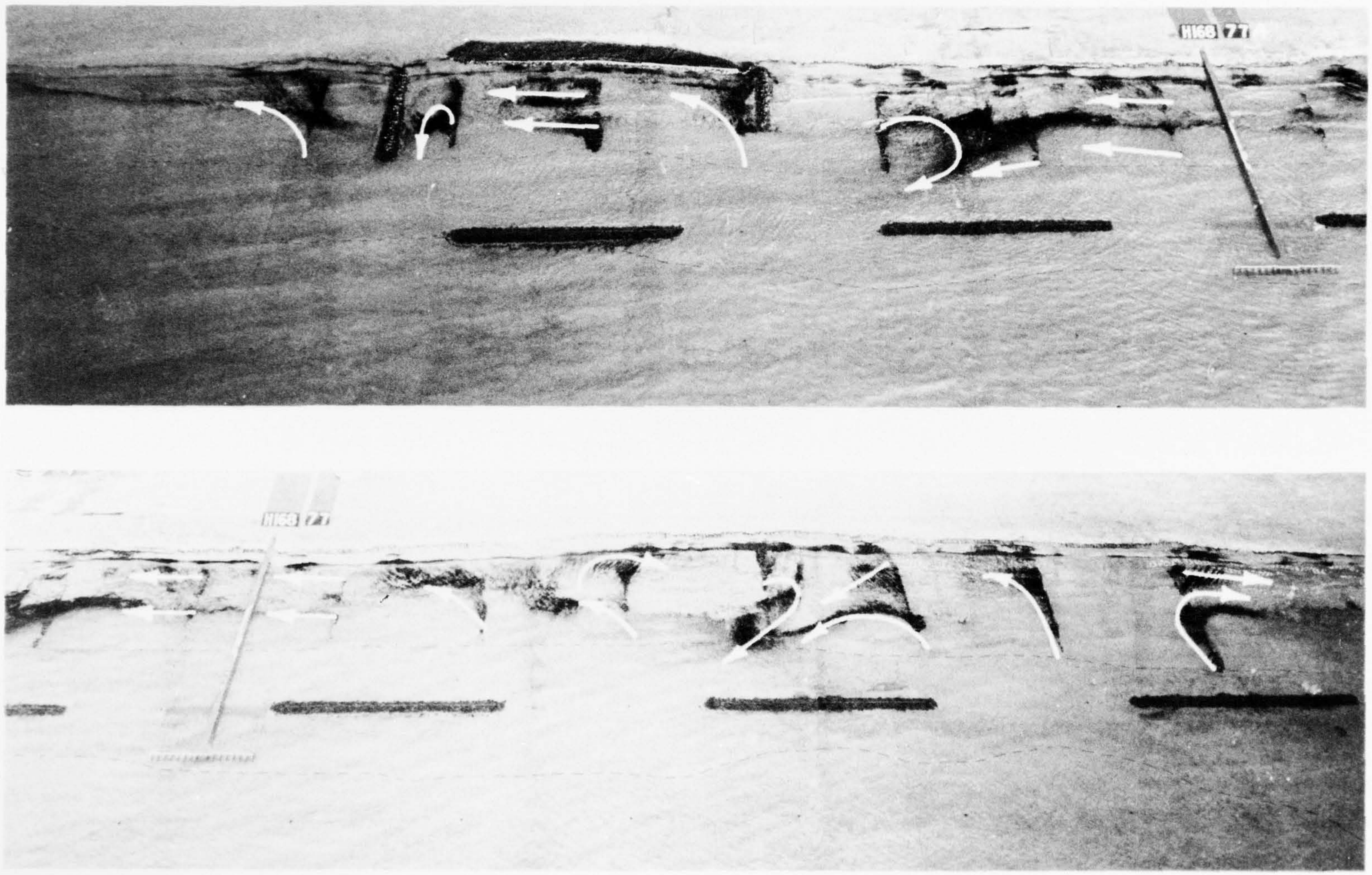


Photo 32. Typical tracer movement for Plan 1 resulting from 14-sec, 7-ft waves from S60°W at mhhw

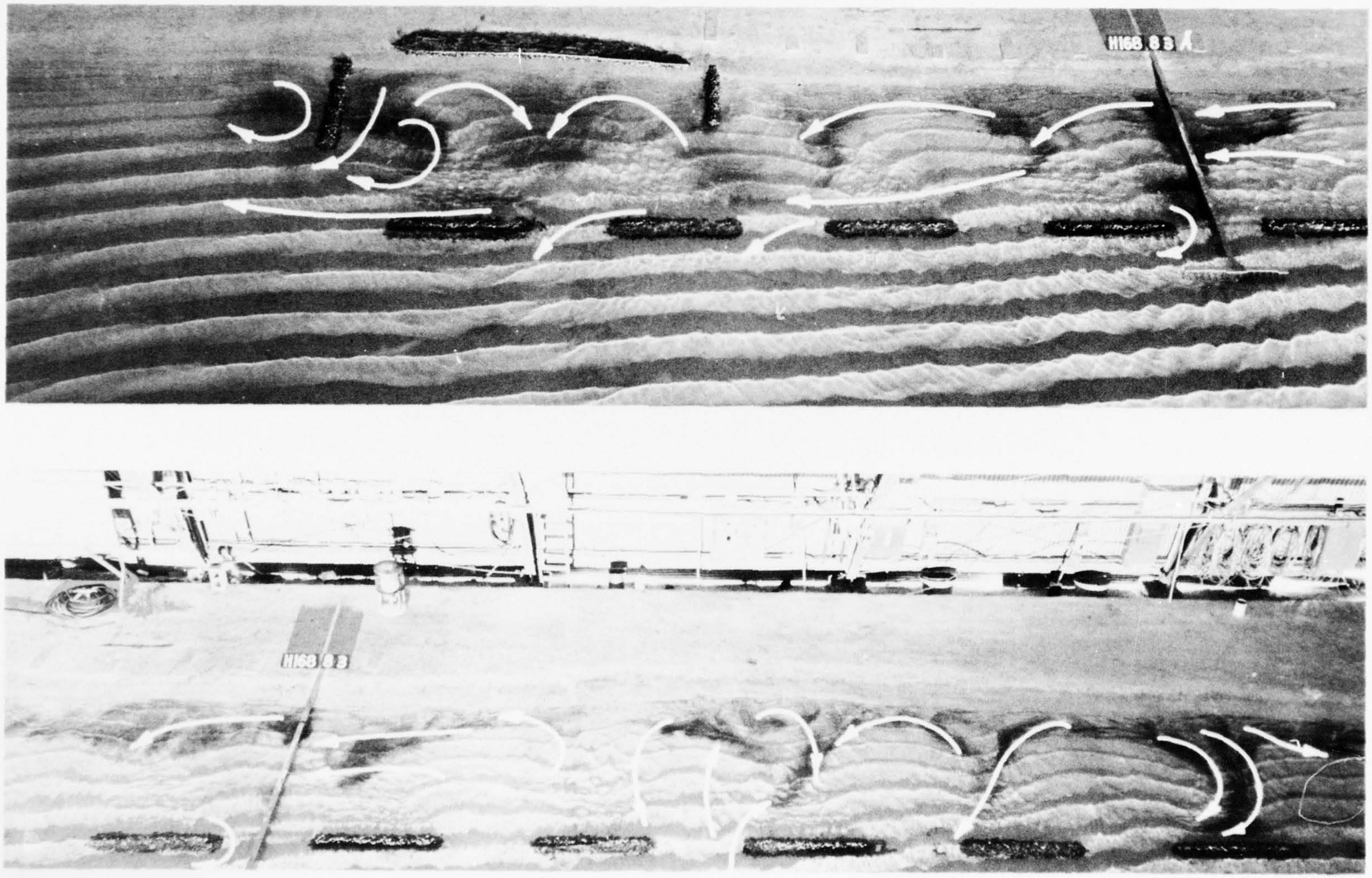


Photo 33. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 2; 7-sec, 11-ft waves from S60°W at mllw

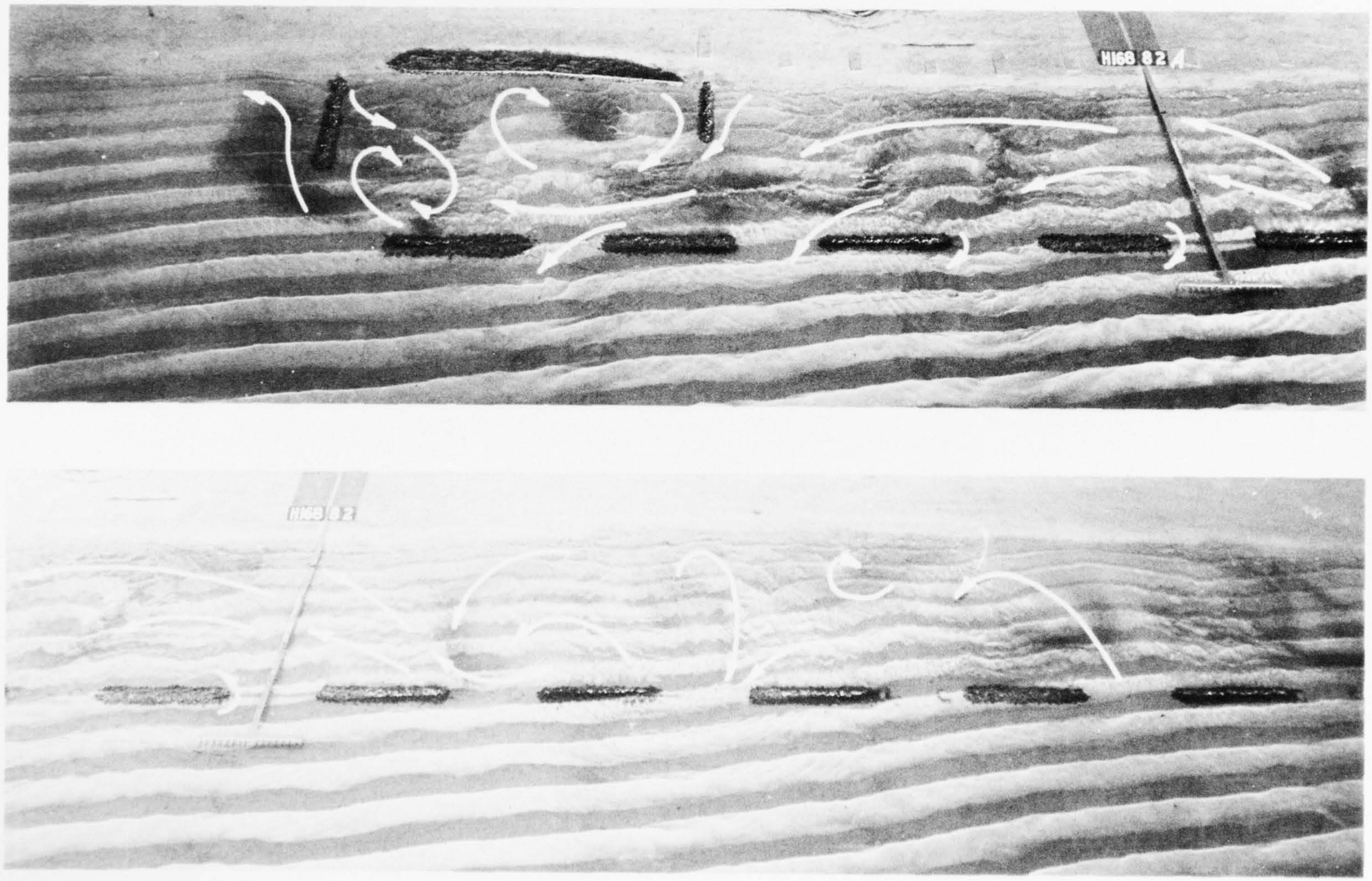


Photo 34. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 2; 7-sec, 11-ft waves from S60°W at mhhw

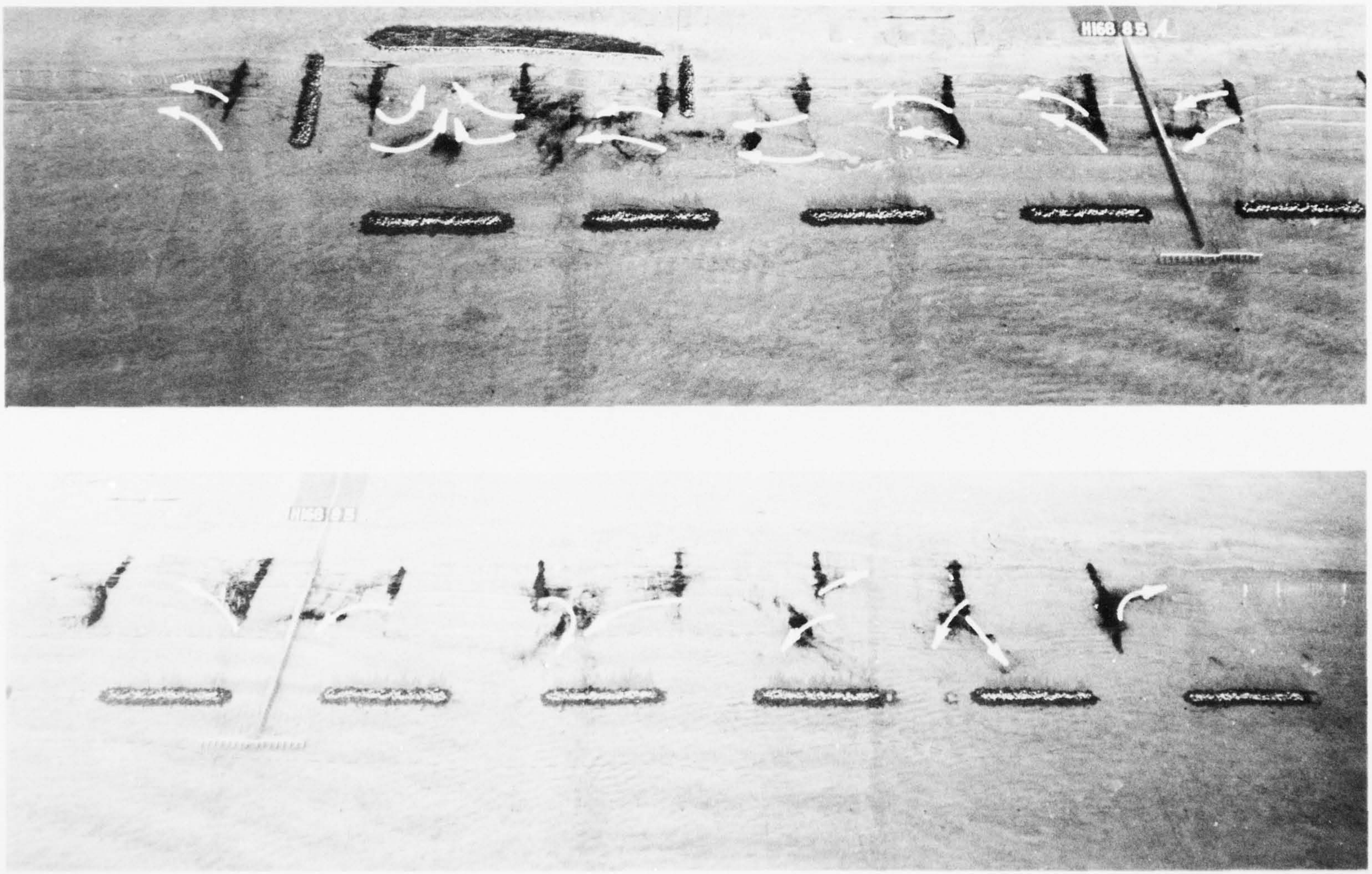


Photo 35. Typical tracer movement for Plan 2 resulting from 7-sec, 11-ft waves from S60°W at mllw

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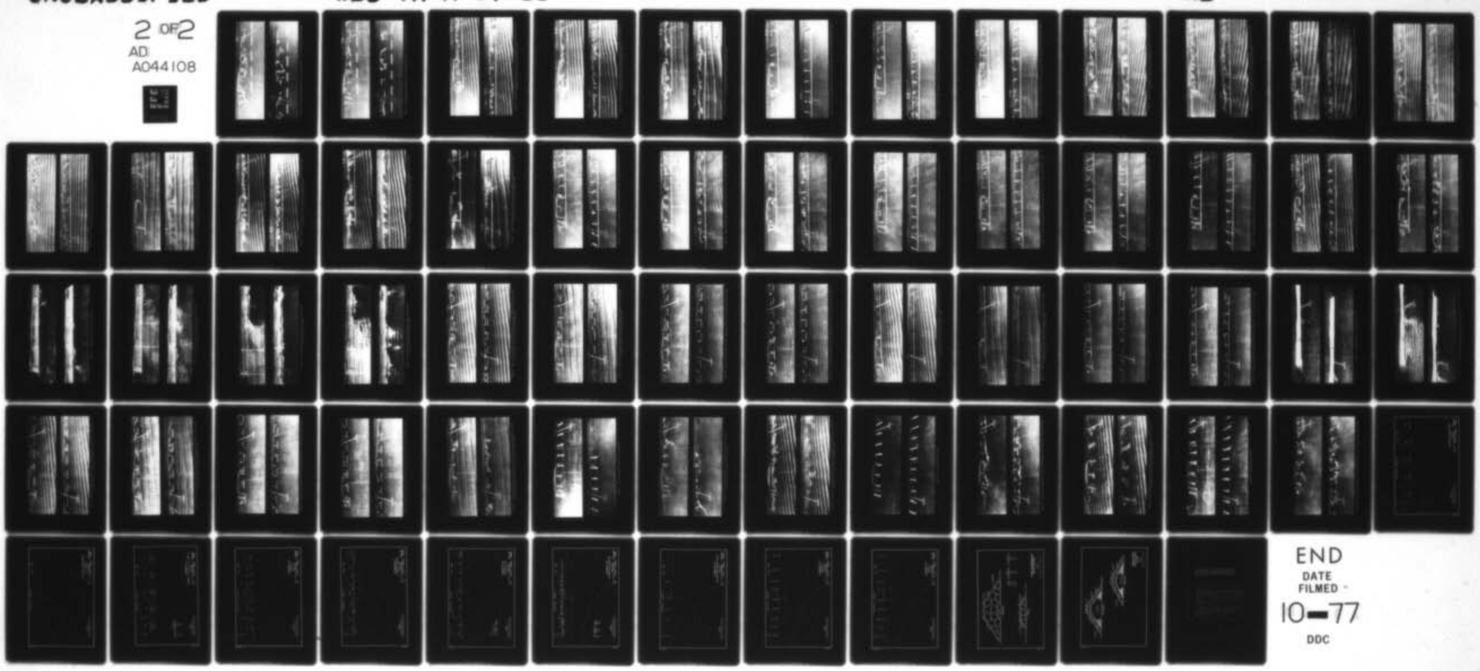

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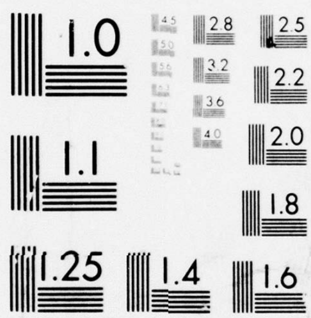

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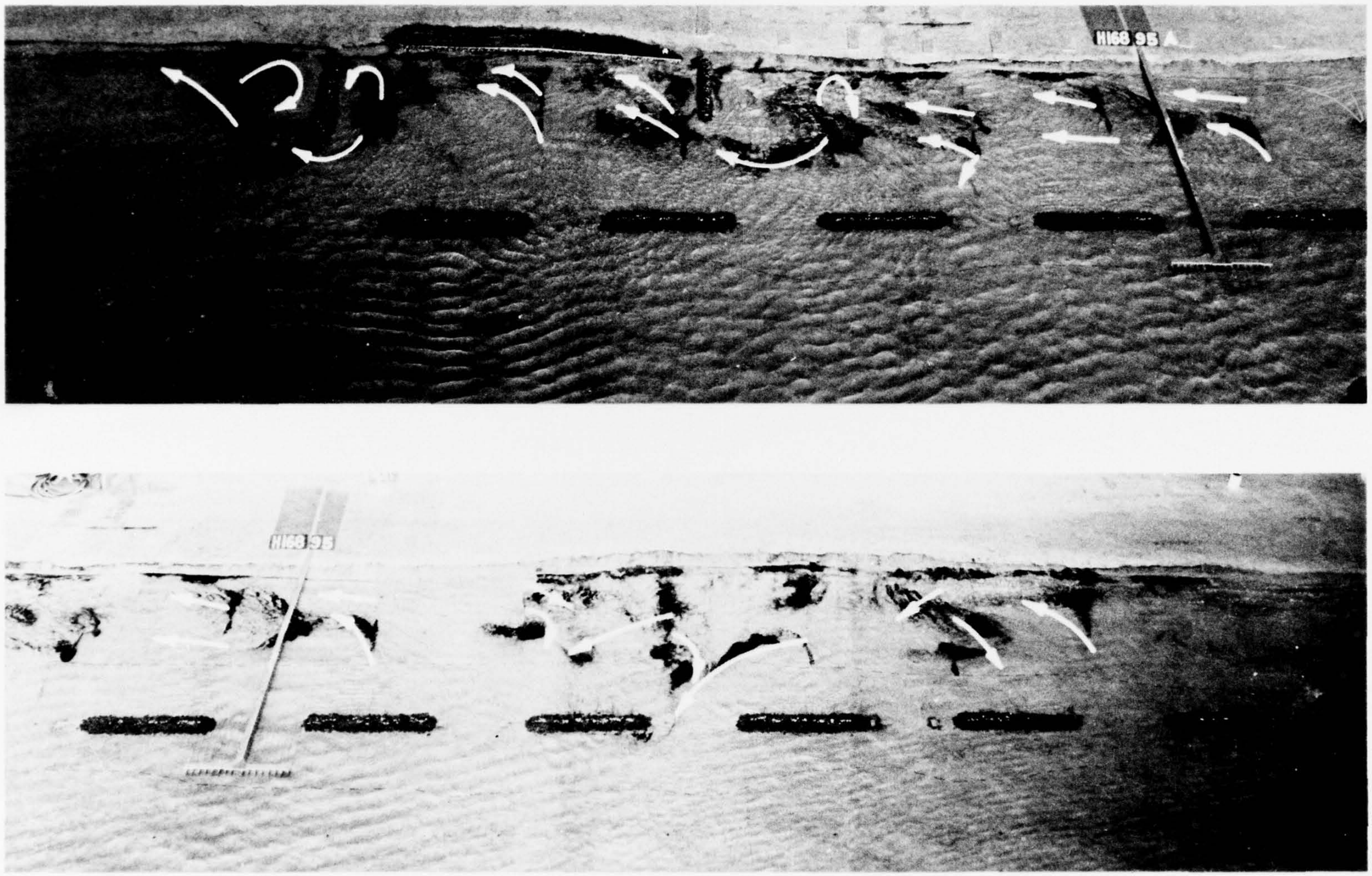


Photo 36. Typical tracer movement for Plan 2 resulting from 7-sec, 11-ft waves from S60°W at mhhw

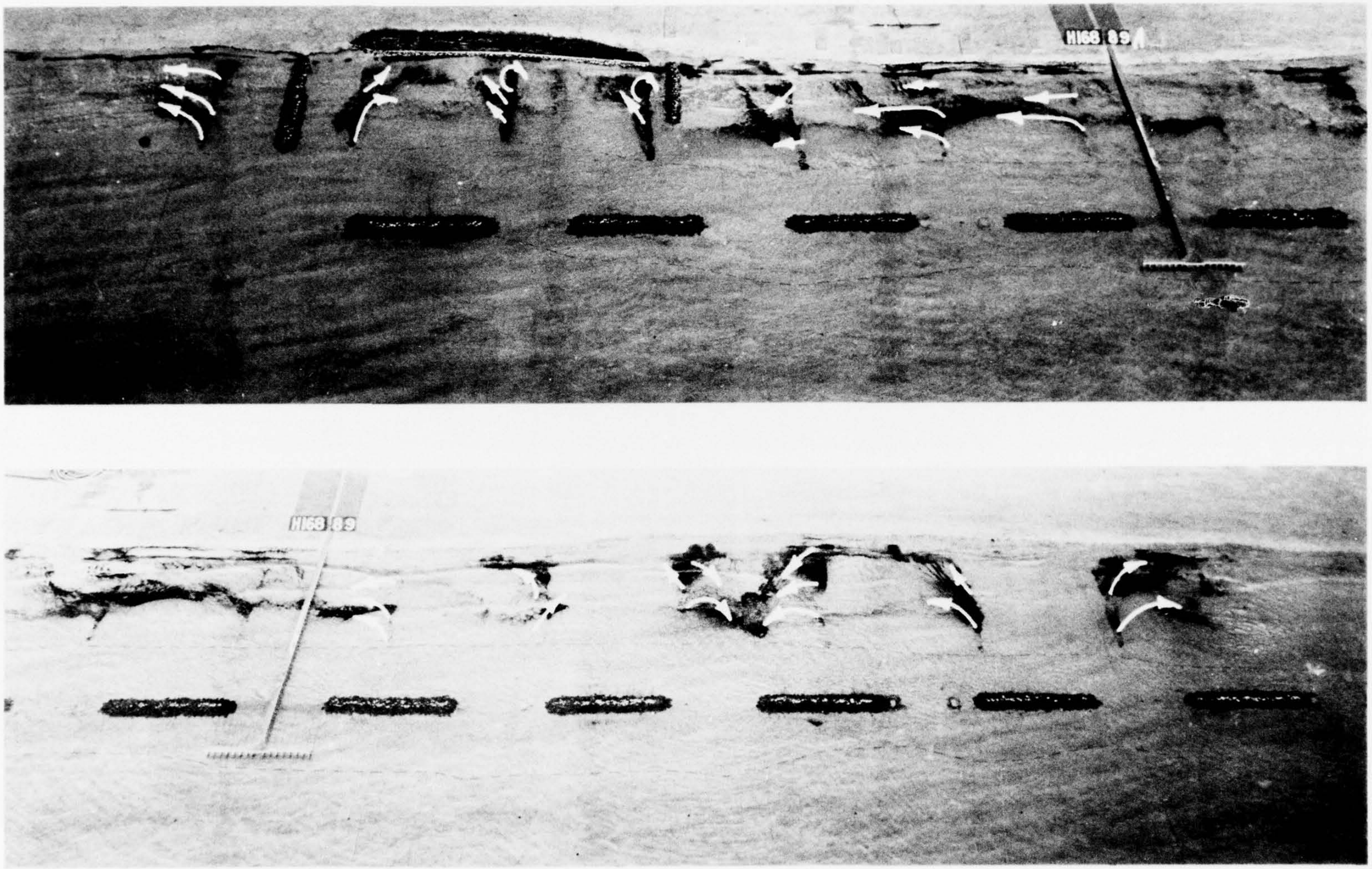


Photo 37. Typical tracer movement for Plan 2 resulting from 14-sec, 7-ft waves from S60°W at mhhw

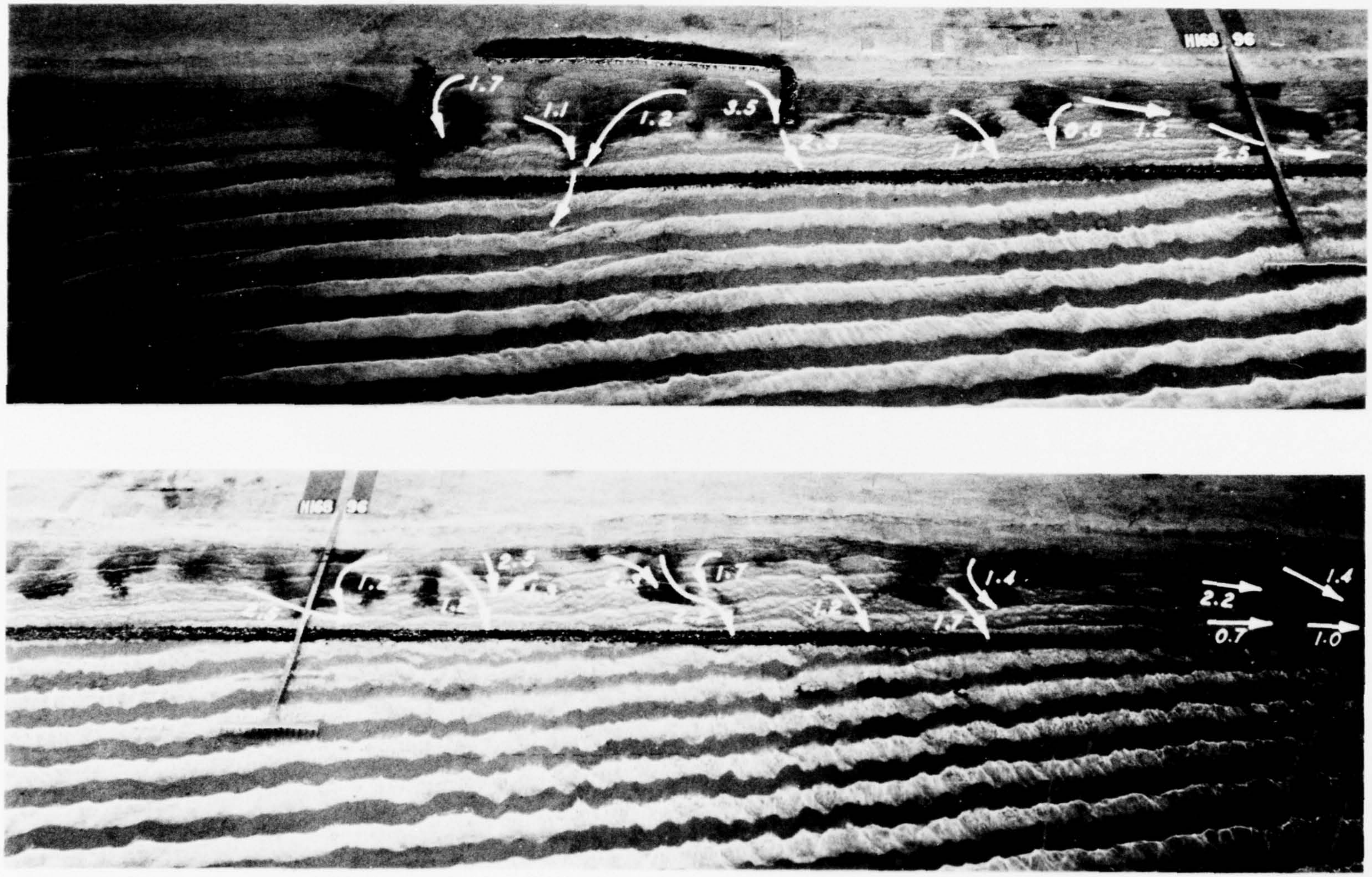
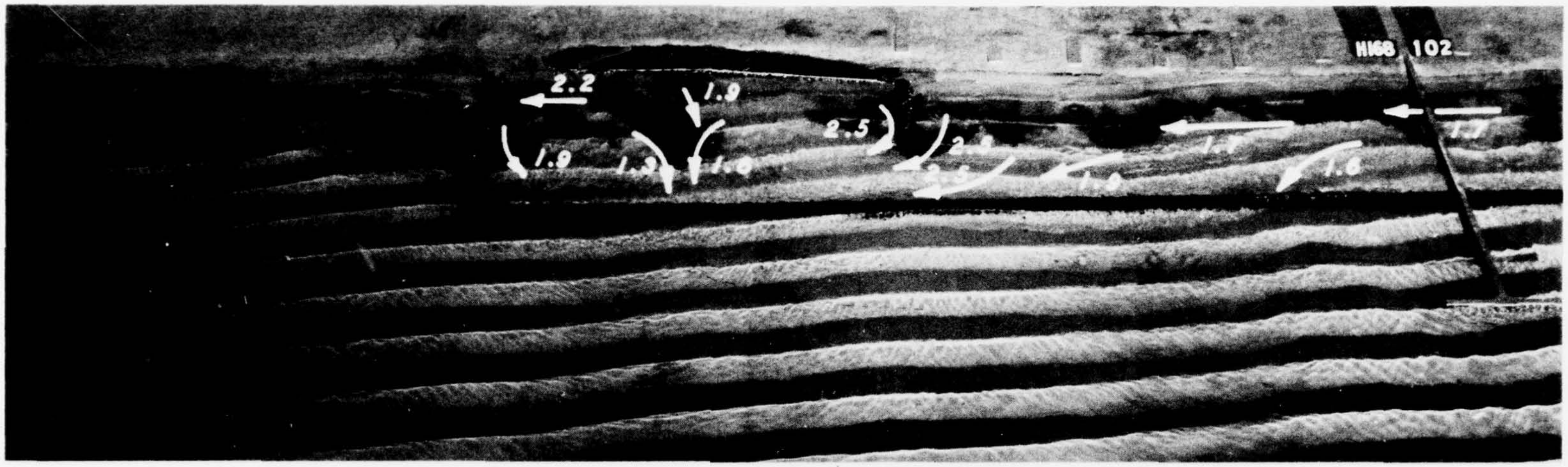


Photo 38. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 3A; 7-sec, 11-ft waves from S60°W at mllw



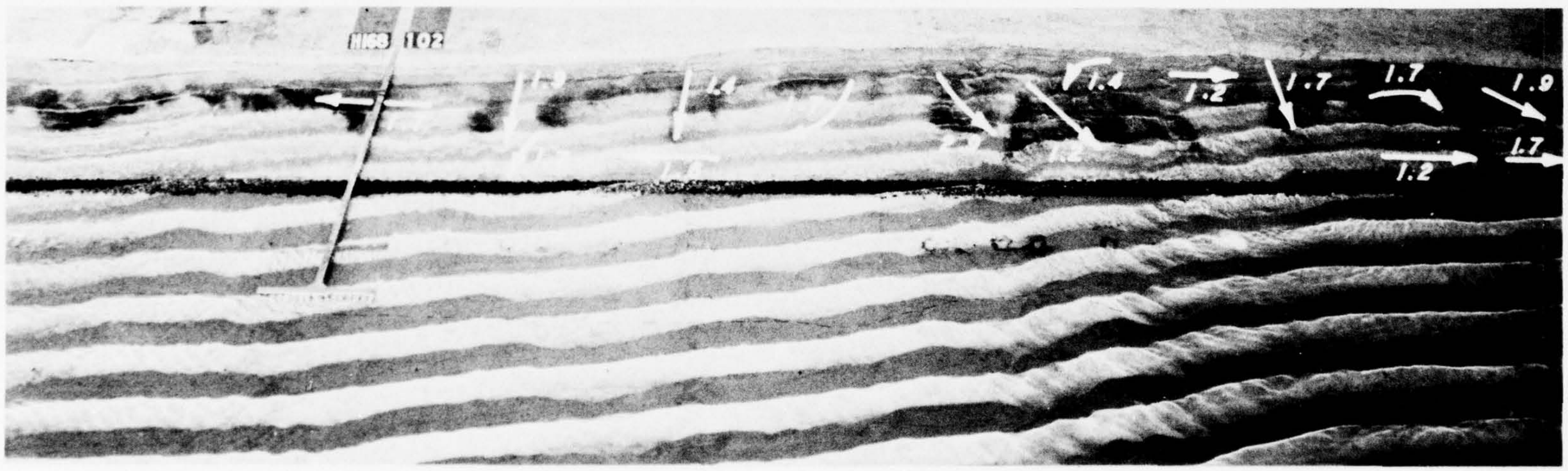


Photo 39. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 3A; 7-sec, 11-ft waves from S60°W at mhhw

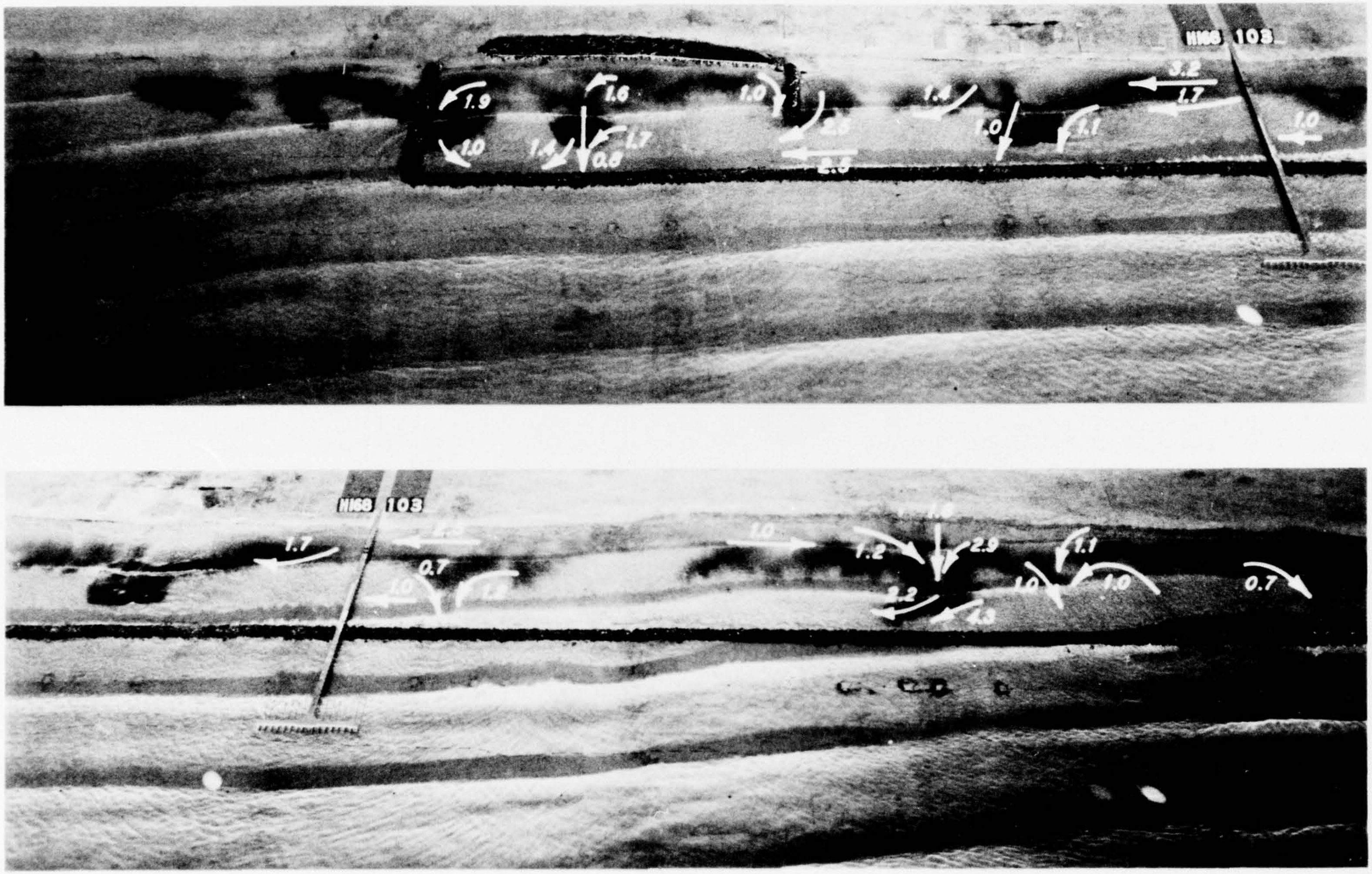


Photo 40. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 3A; 14-sec, 7-ft waves from S60°W at mhhw

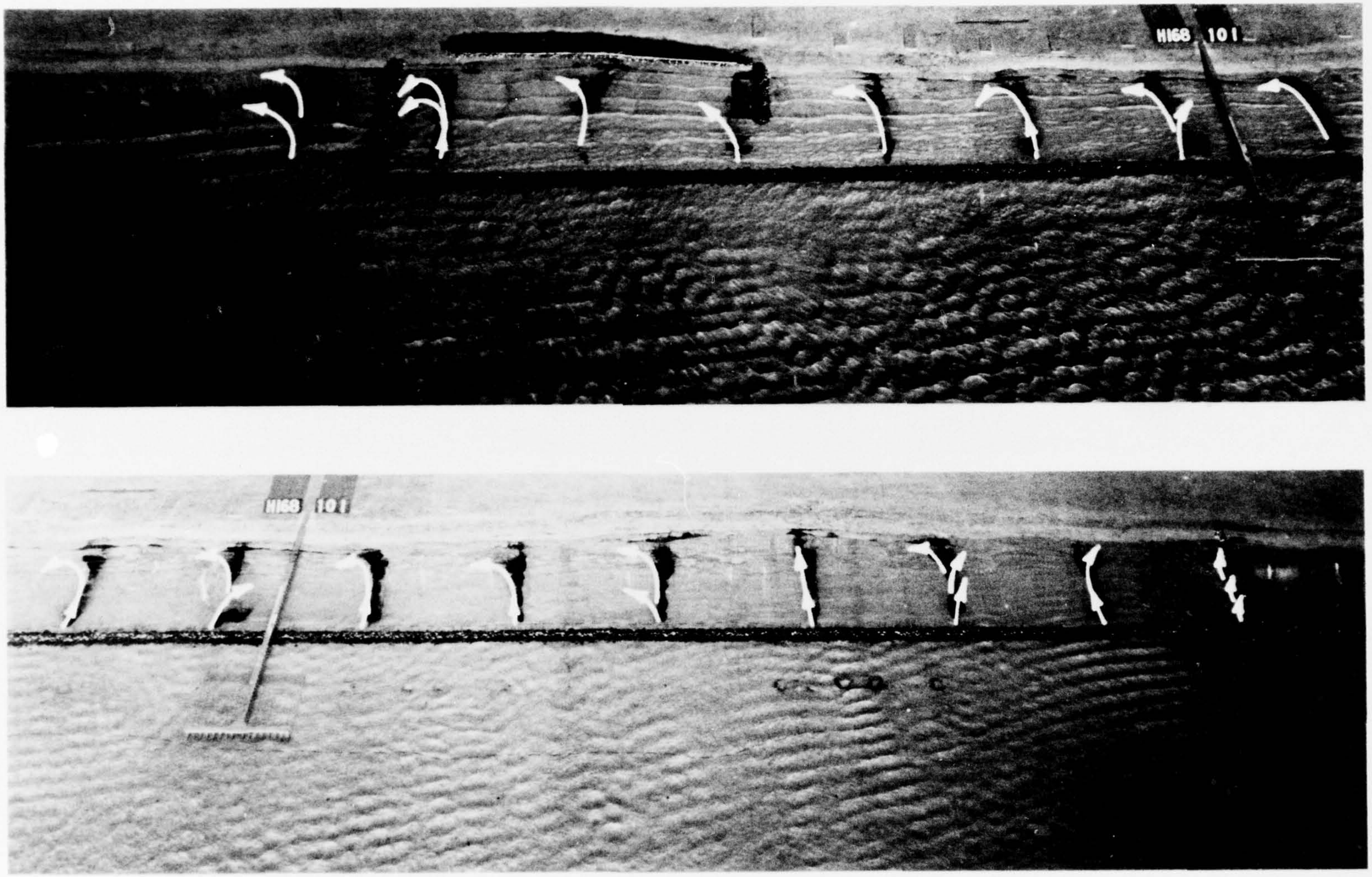


Photo 41. Typical tracer movement for Plan 3A resulting from 7-sec, 11-ft waves from S60°W at mllw

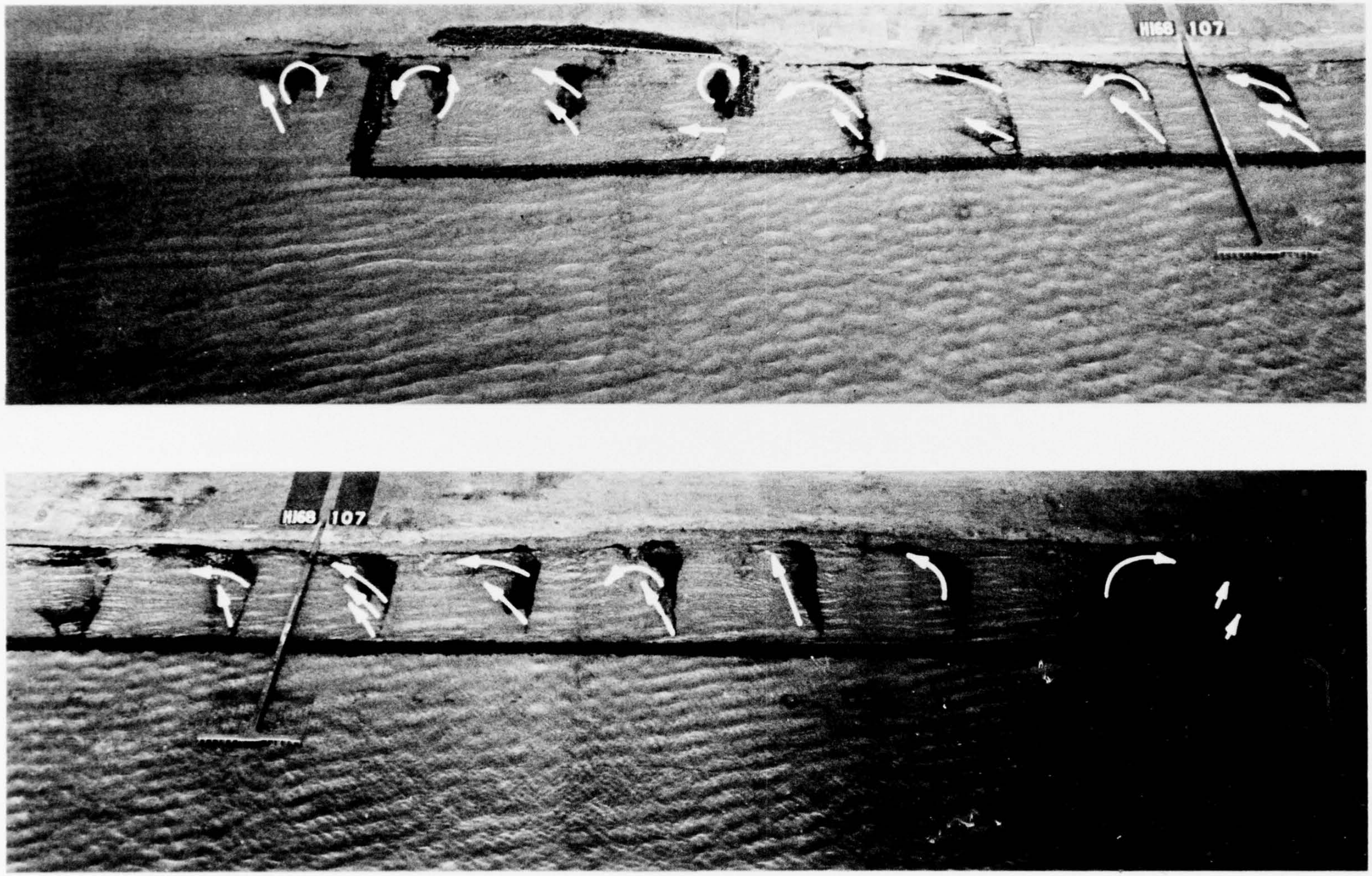


Photo 42. Typical tracer movement for Plan 3A resulting from 7-sec, 11-ft waves from S60°W at mhhw

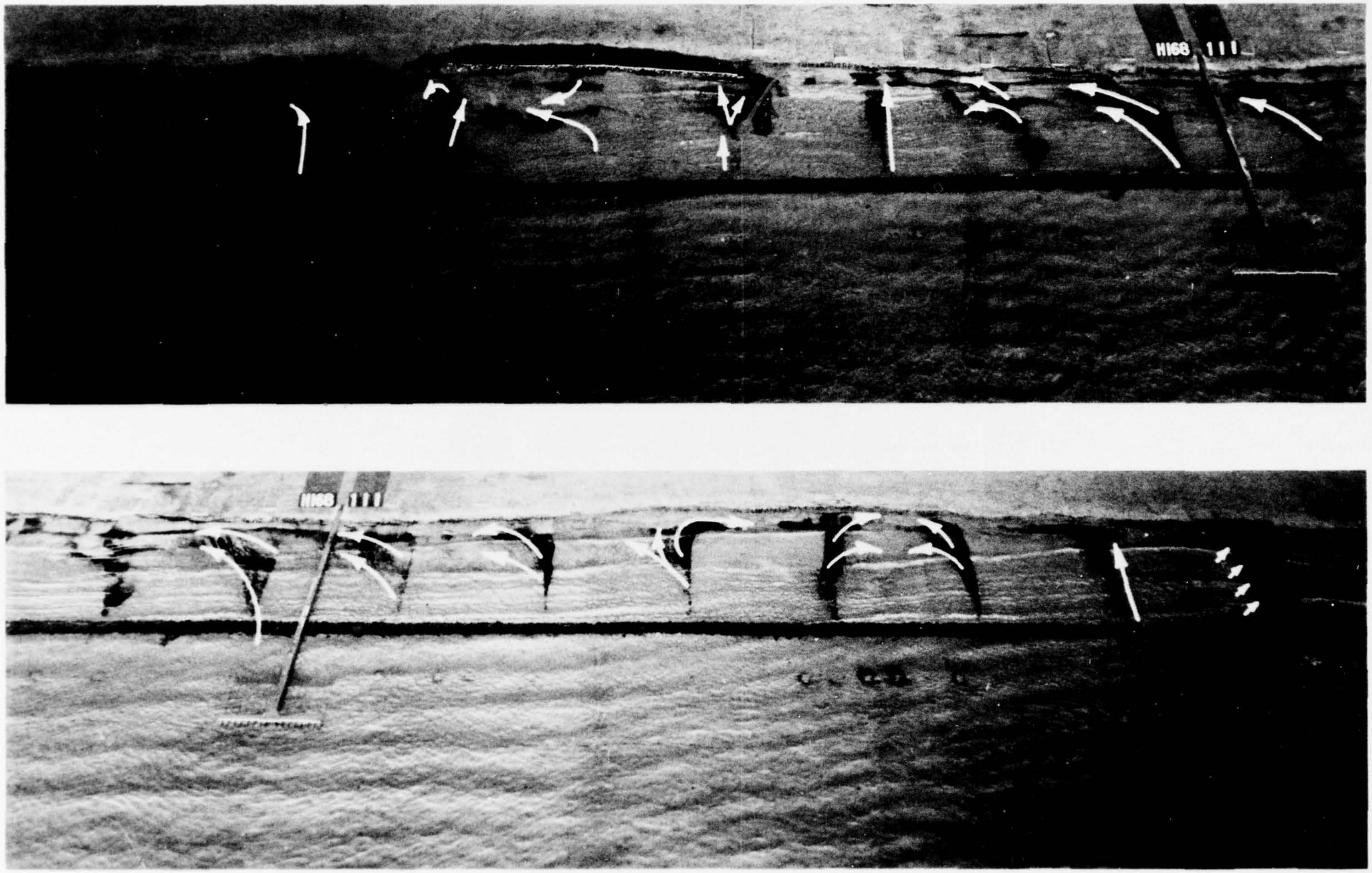


Photo 43. Typical tracer movement for Plan 3A resulting from 14-sec, 7-ft waves from S60°W at mhhw

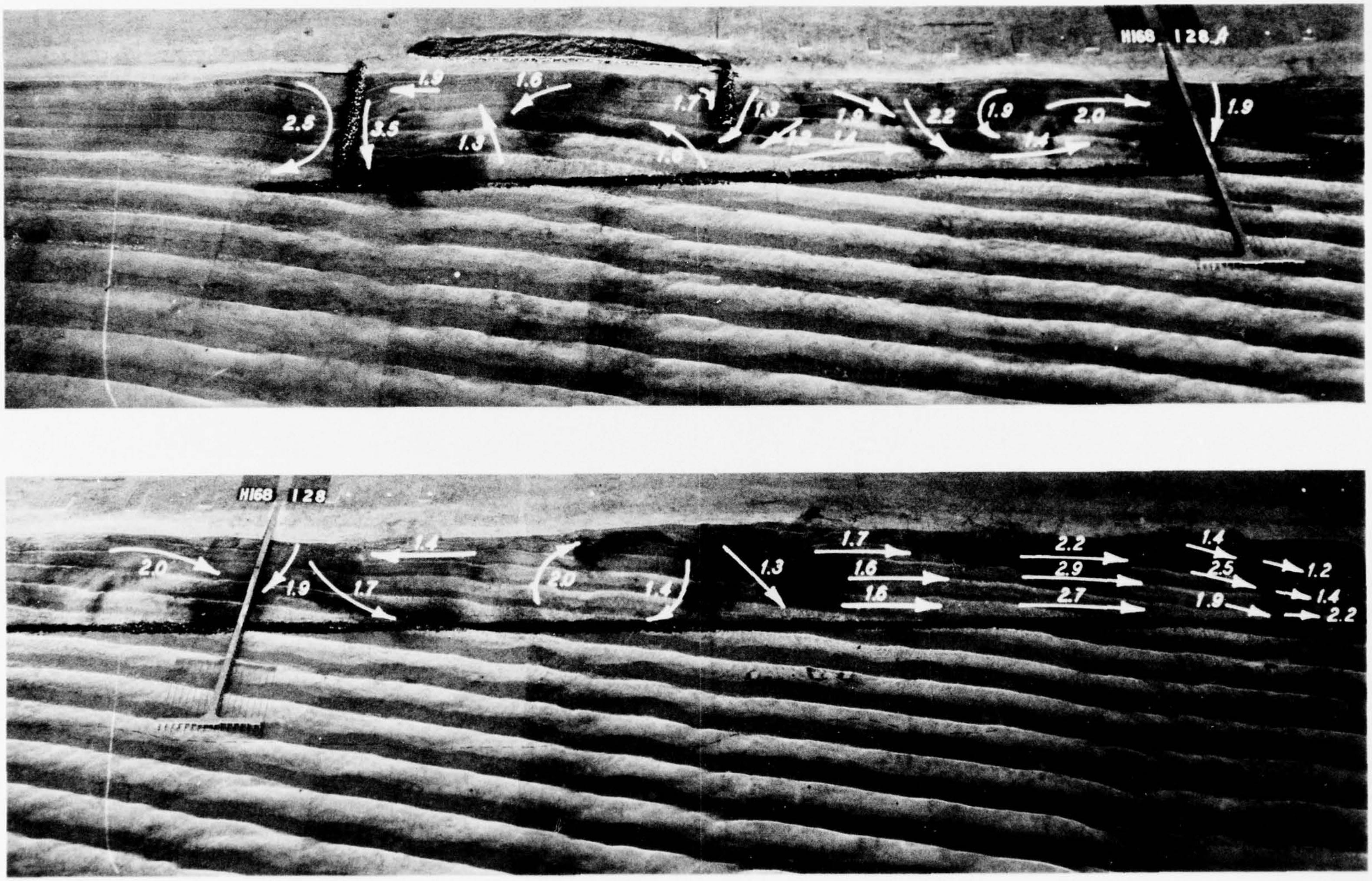


Photo 44. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 7-sec, 7-ft waves from N60°W at mllw

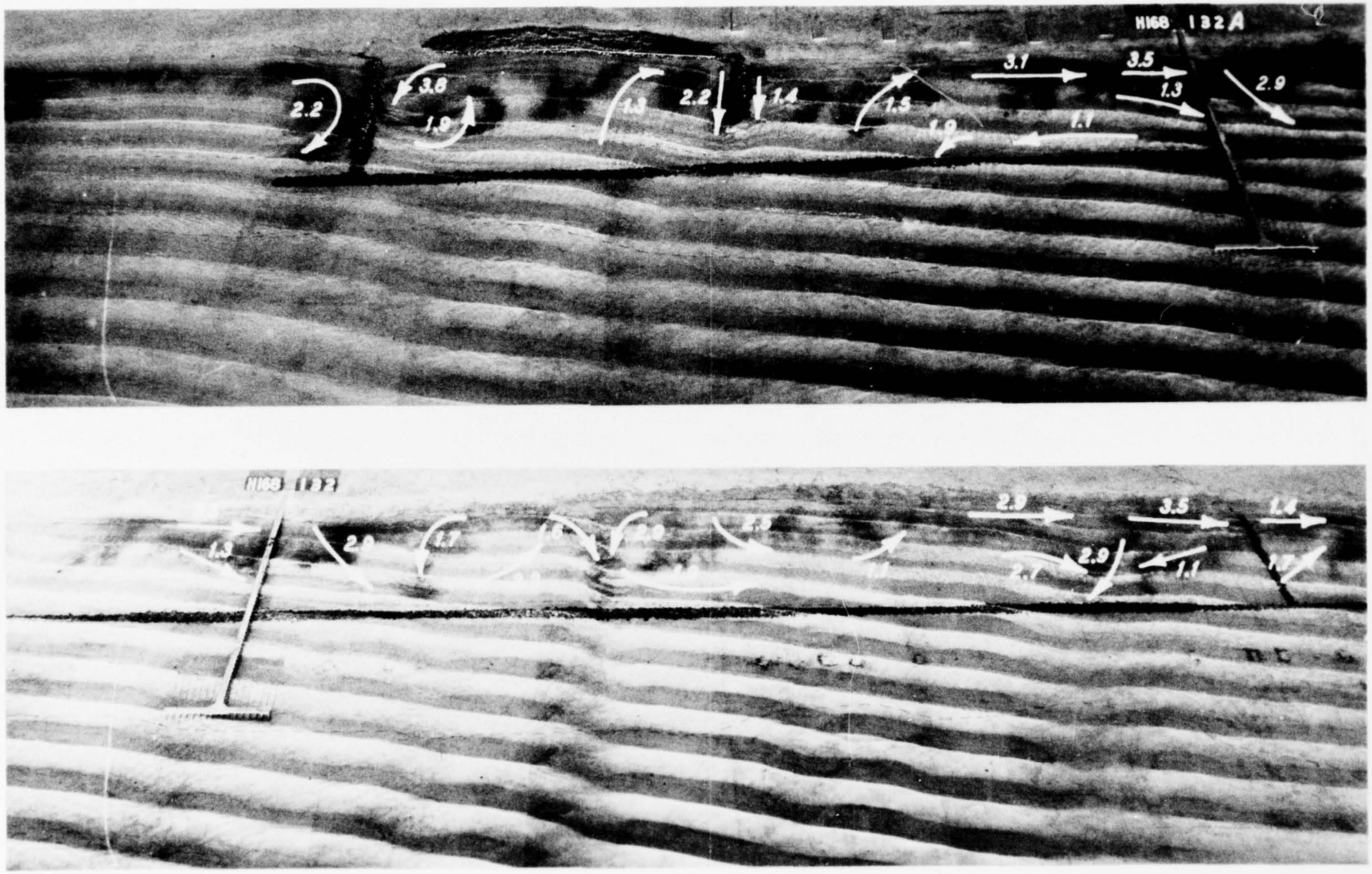


Photo 45. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 7-sec, 7-ft waves from N60°W at mhhw

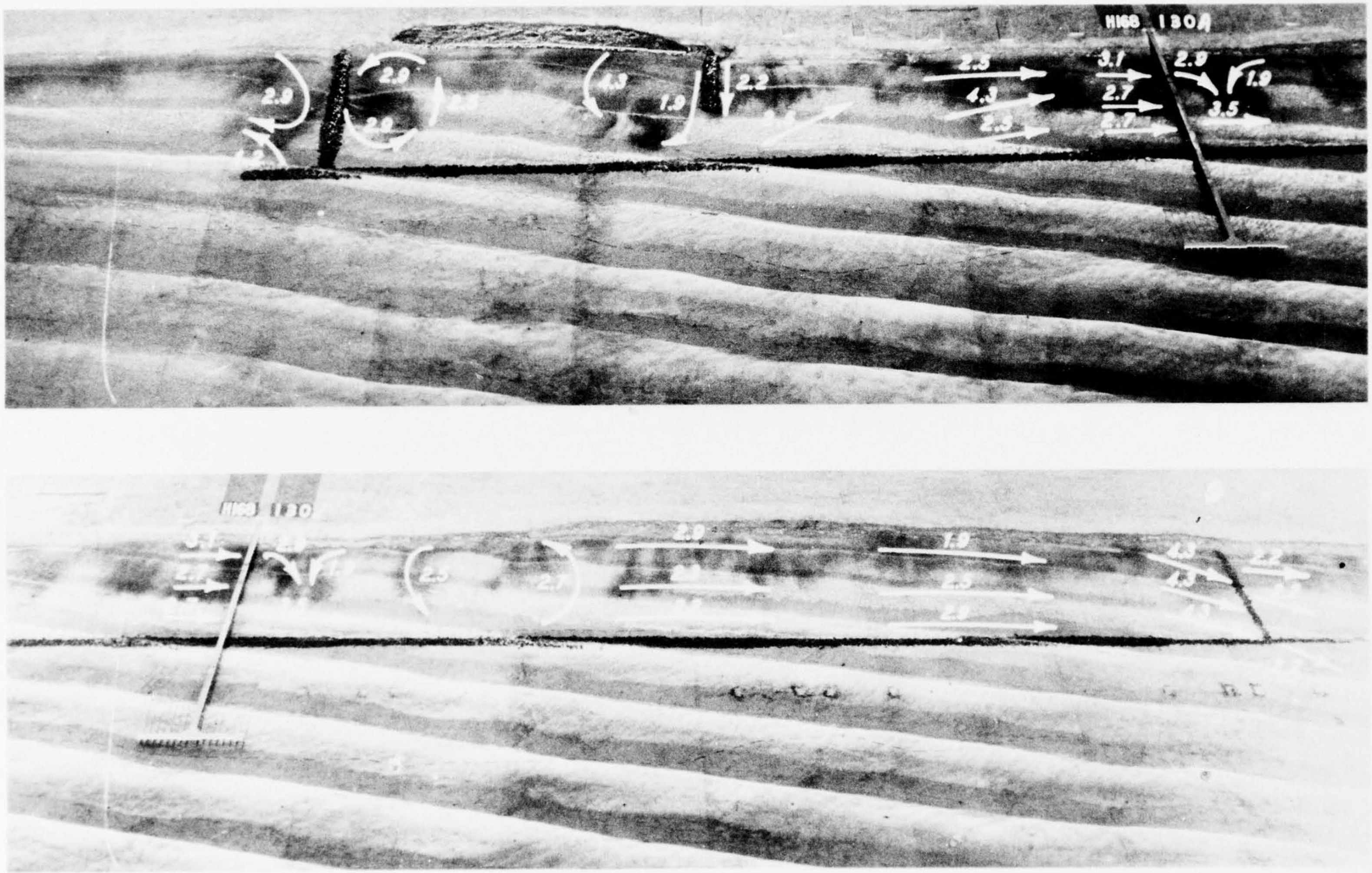


Photo 46. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 10-sec, 9-ft waves from N60°W at mhhw

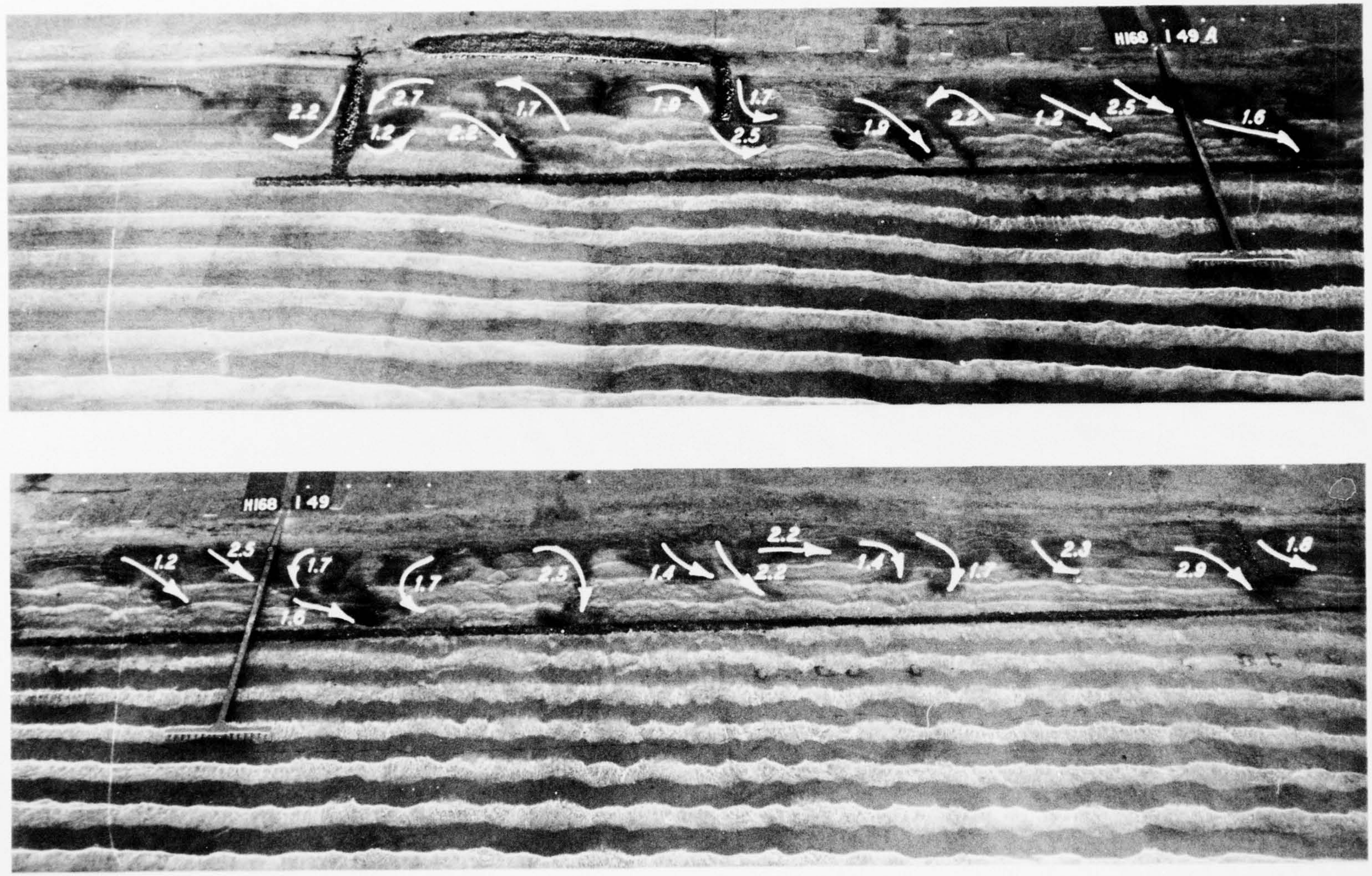
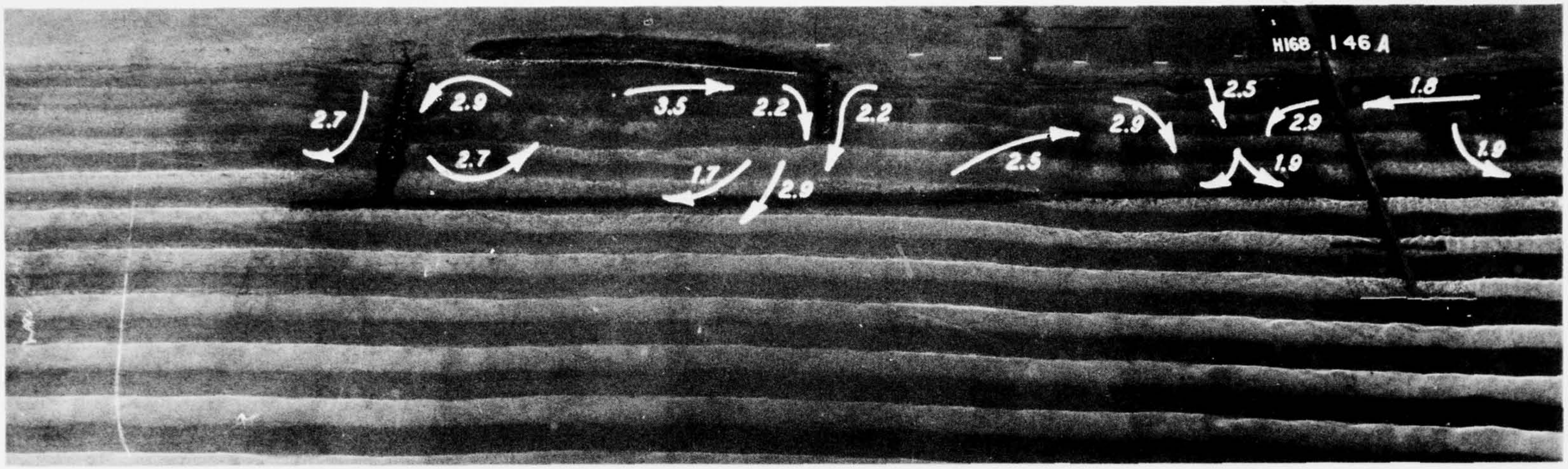


Photo 47. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 7-sec, 11-ft waves from west at mllw



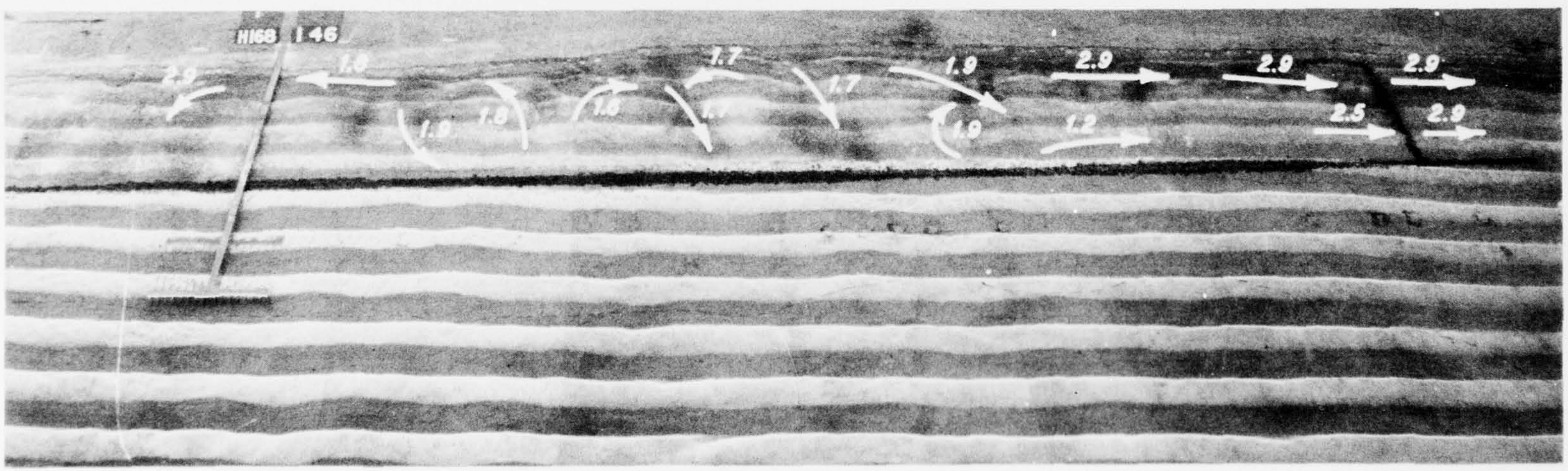


Photo 48. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 7-sec, 11-ft waves from west at mhhw

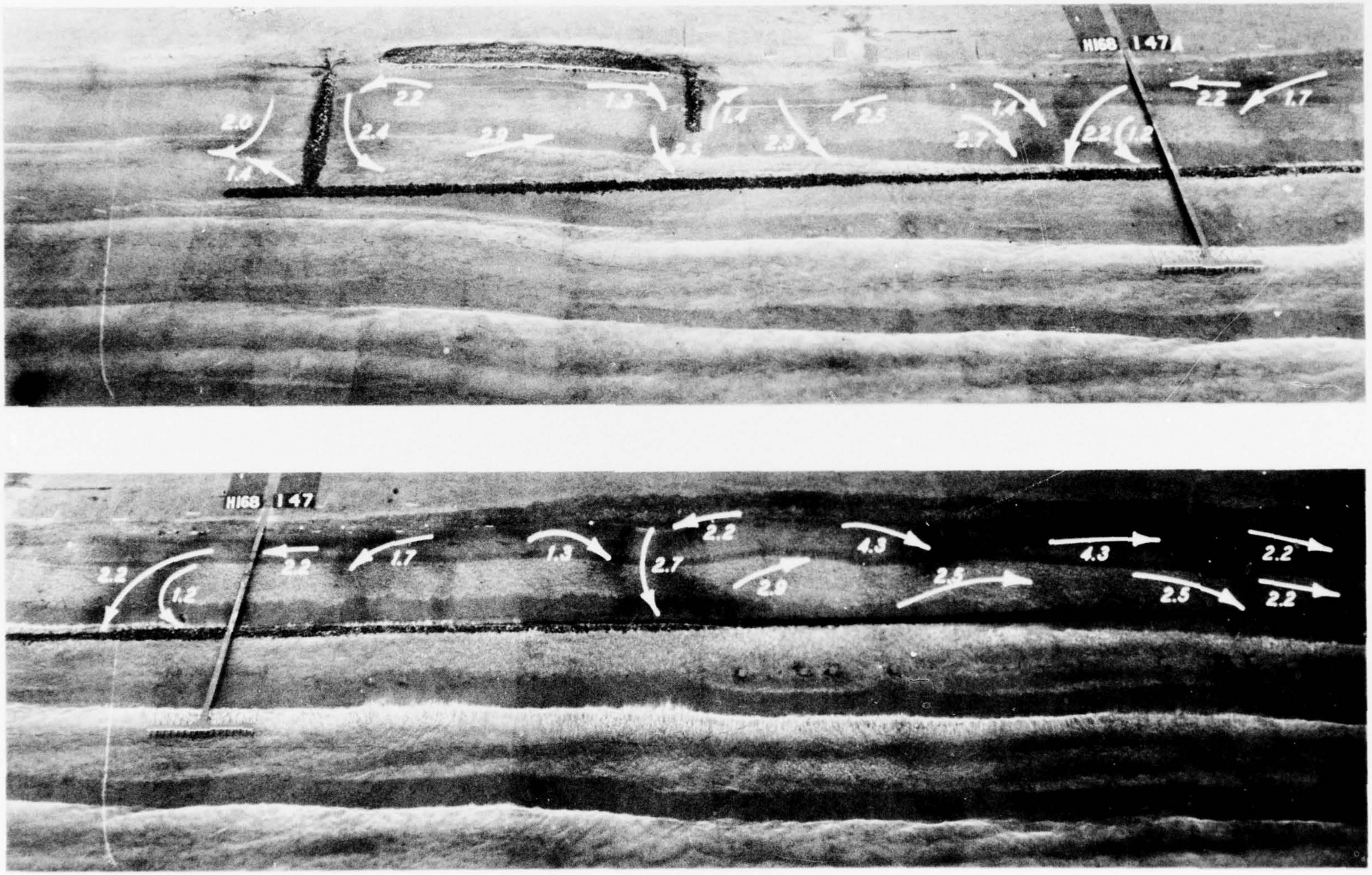
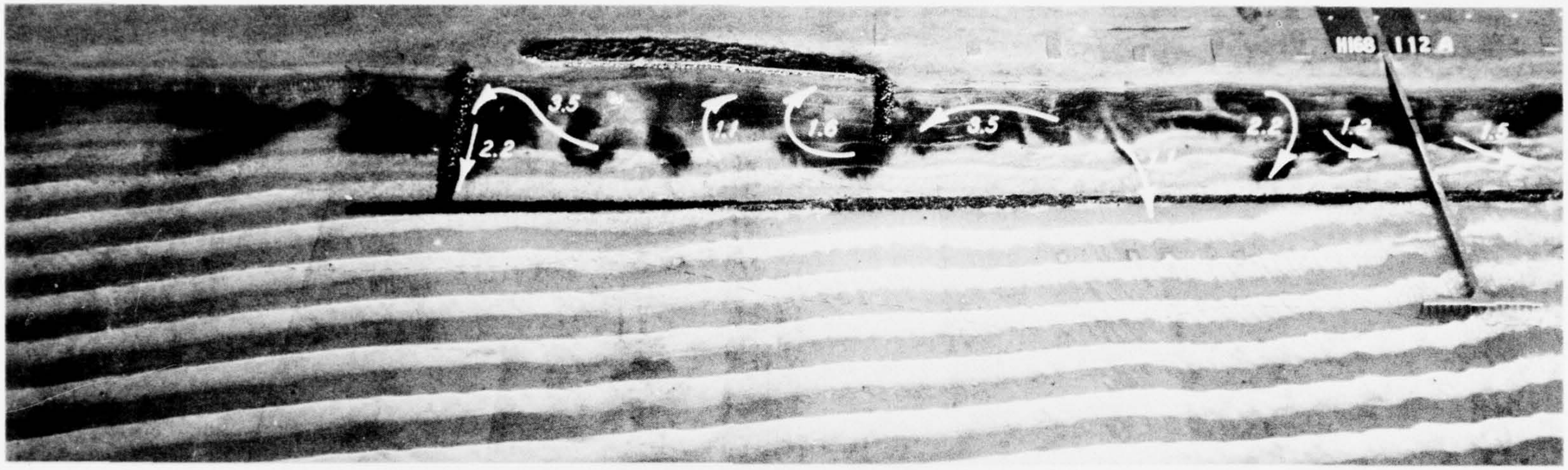


Photo 49. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 14-sec, 9-ft waves from west at mhhw



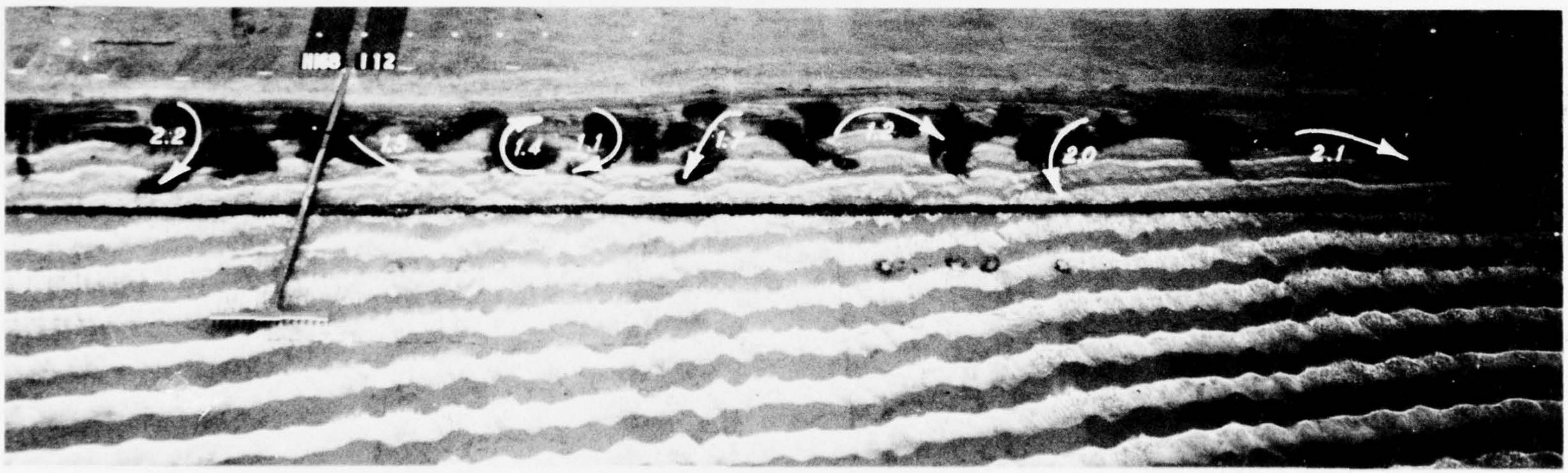


Photo 50. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 7-sec, 11-ft waves from S60°W at mllw

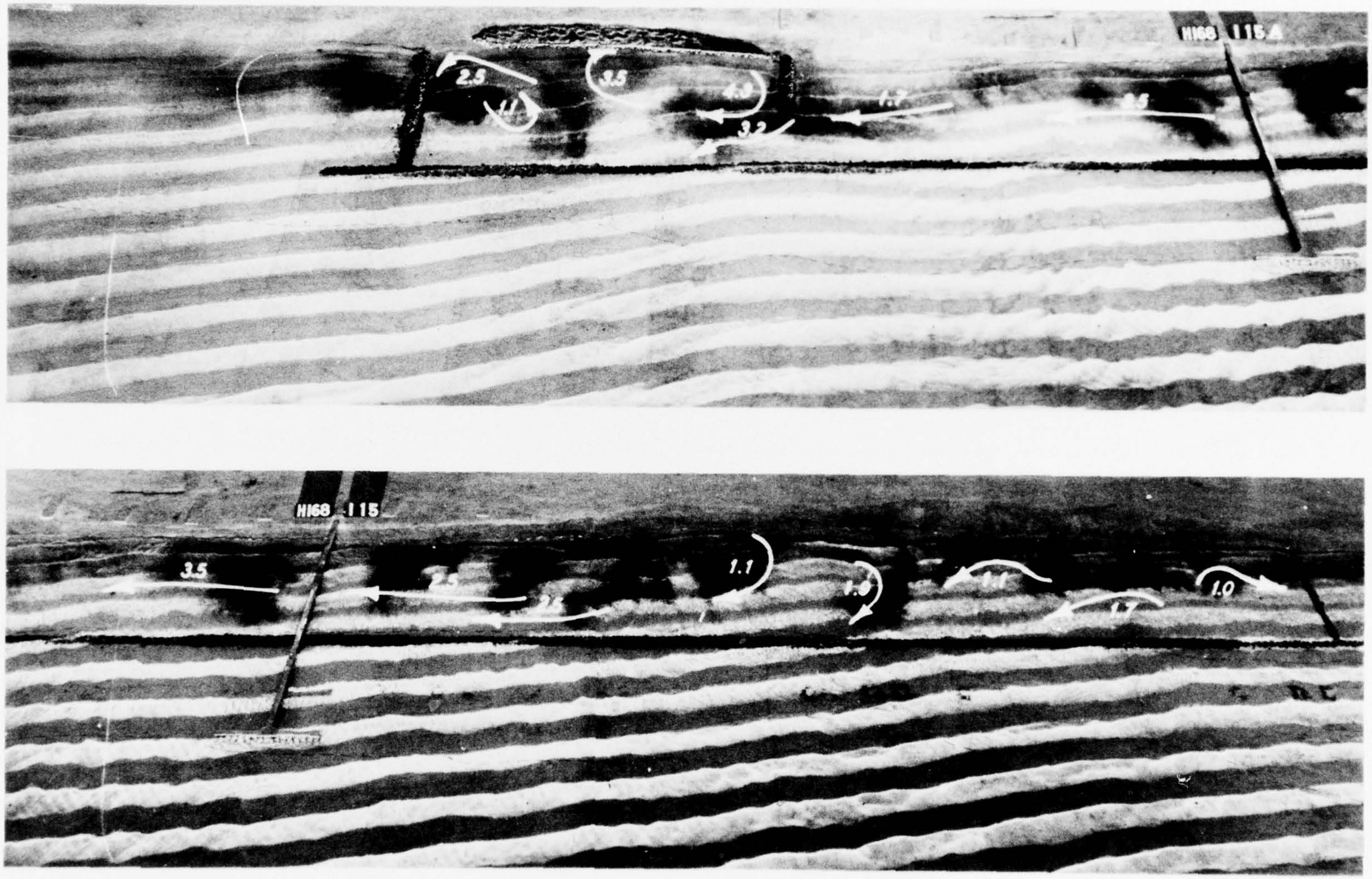


Photo 51. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 7-sec, 11-ft waves from S60°W at mhhw

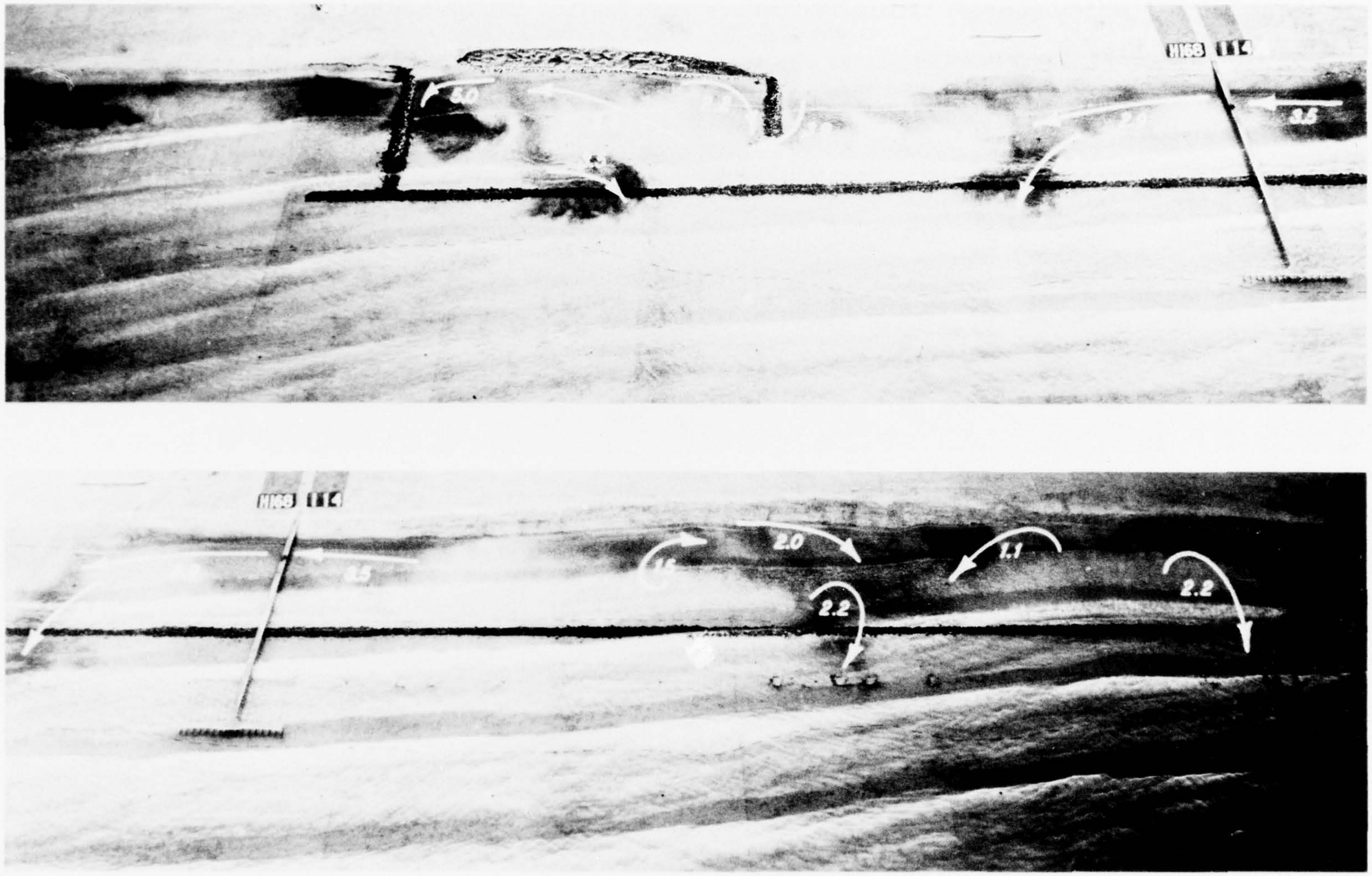


Photo 52. Typicsl wave and current patterns and current magnitudes (prototype feet per second) for Plan 4; 14-sec, 7-ft waves from S60°W at mhhw

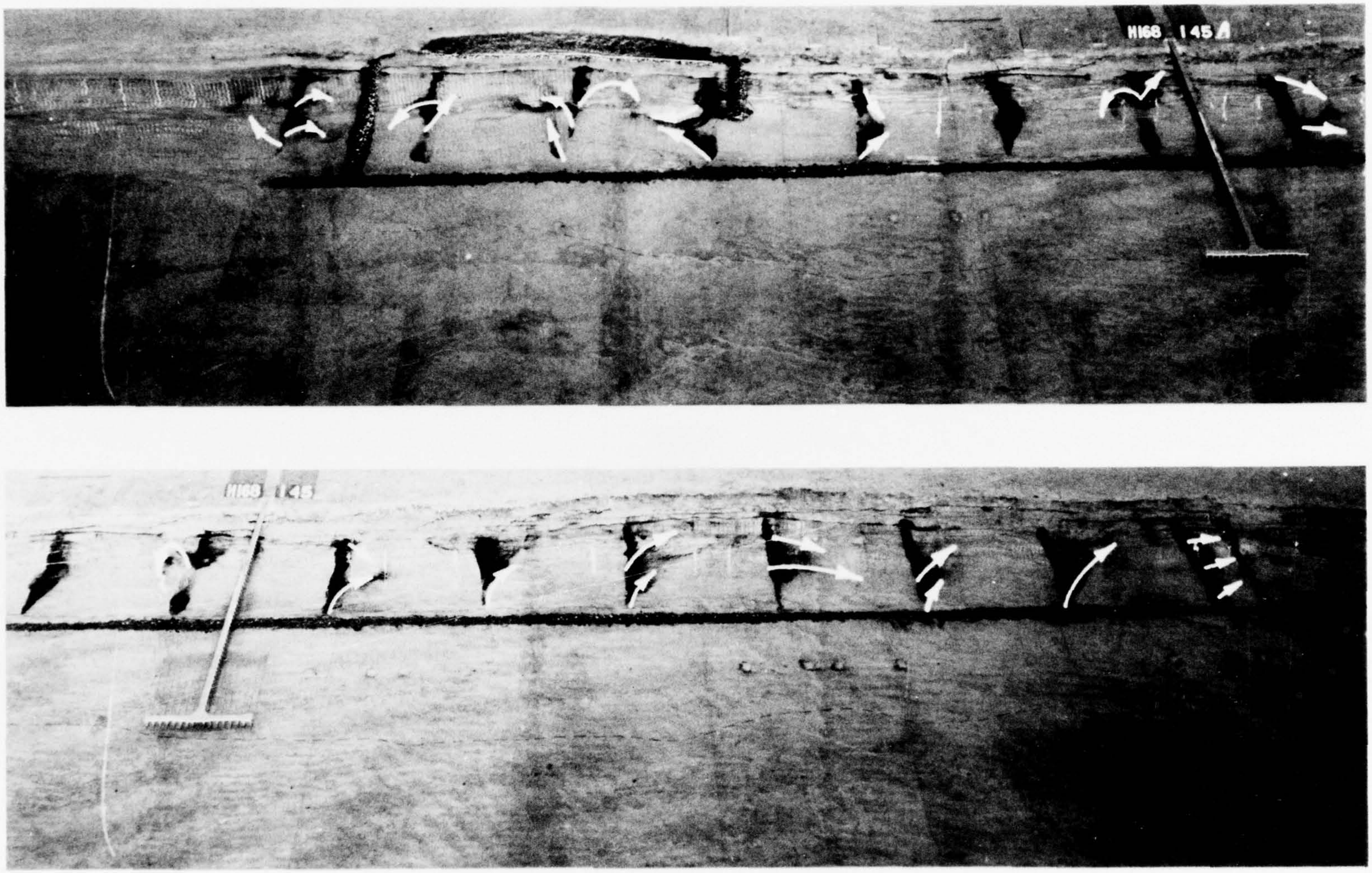


Photo 53. Typical tracer movement for Plan 4 resulting from 7-sec, 7-ft waves from N60°W at mllw

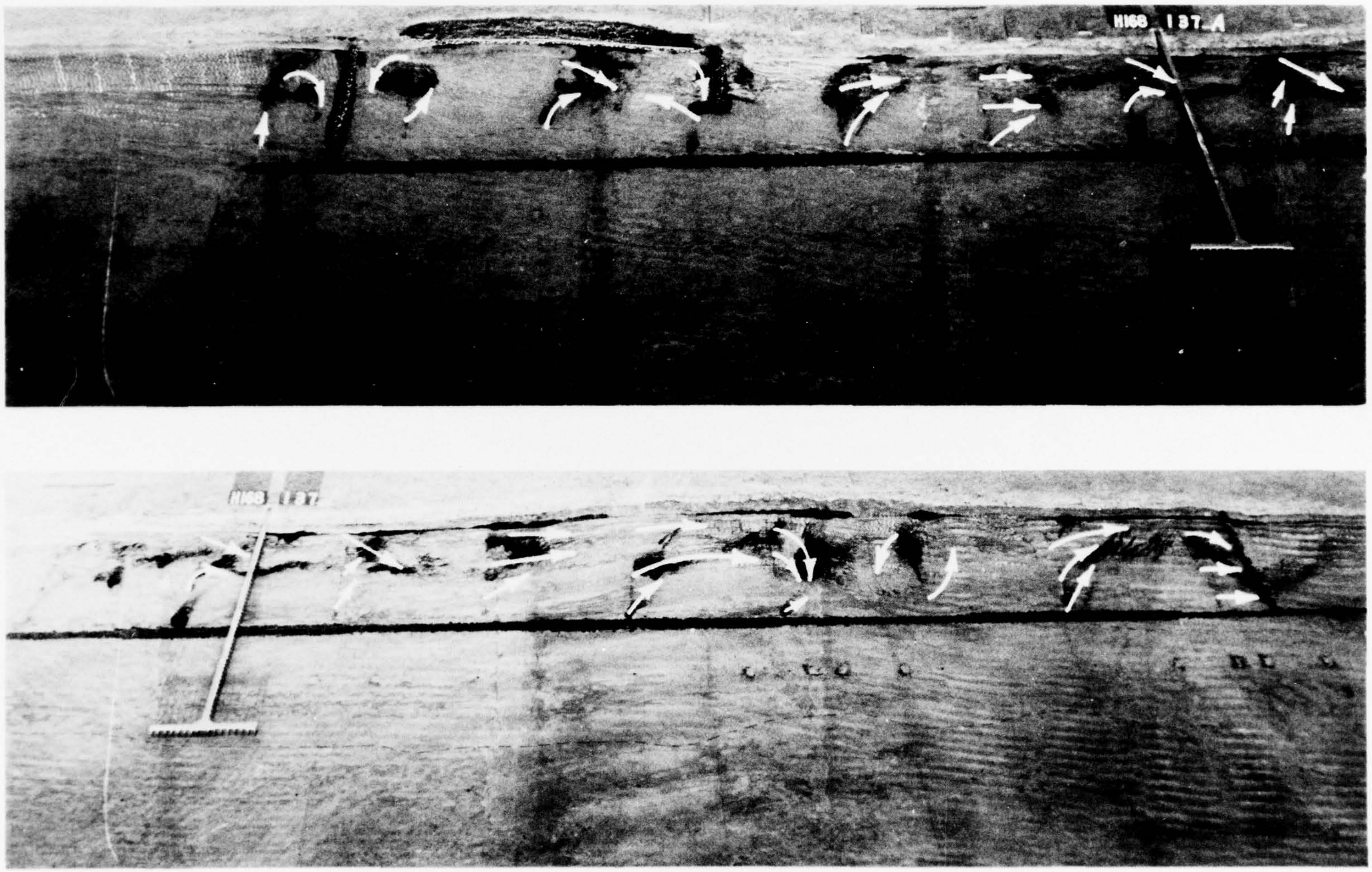
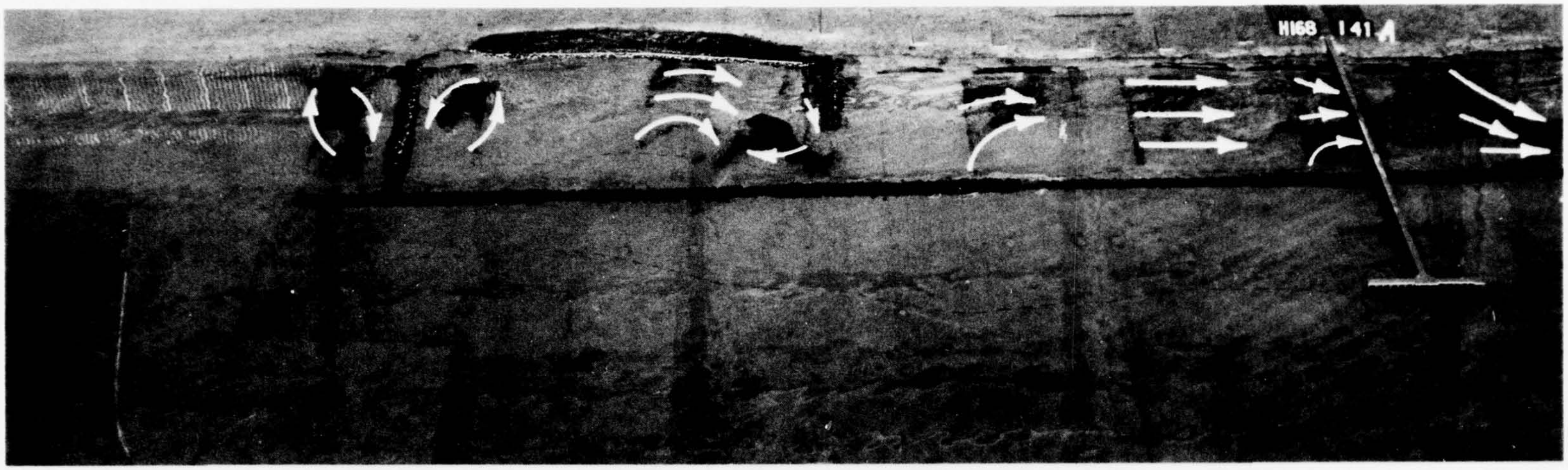


Photo 54. Typical tracer movement for Plan 4 resulting from 7-sec, 7-ft waves from N60°W at mhhw



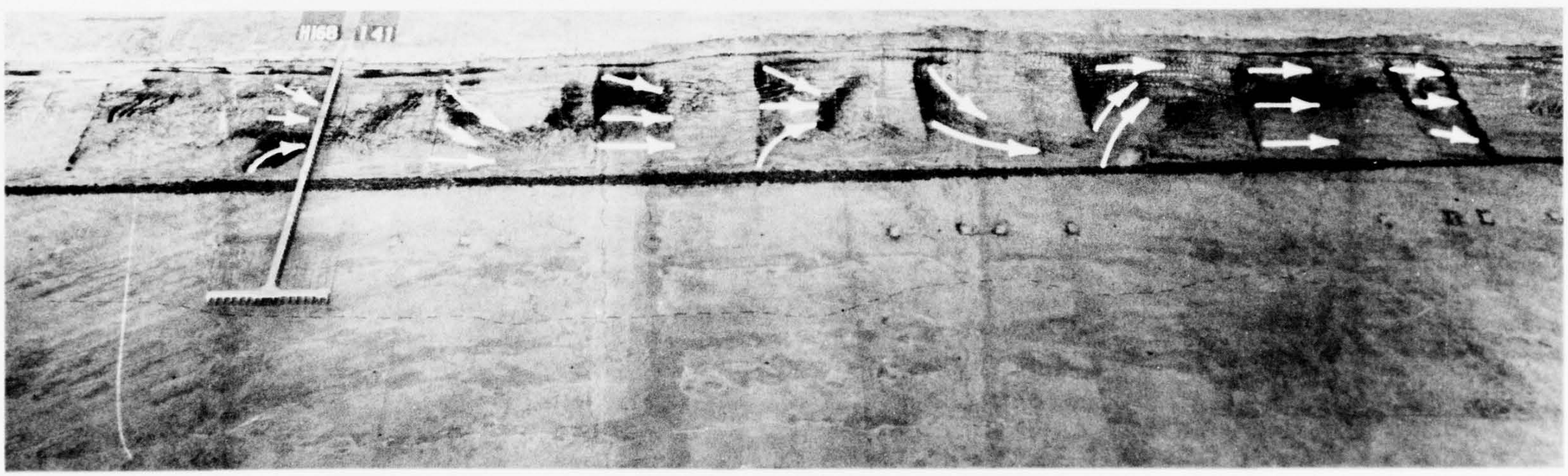
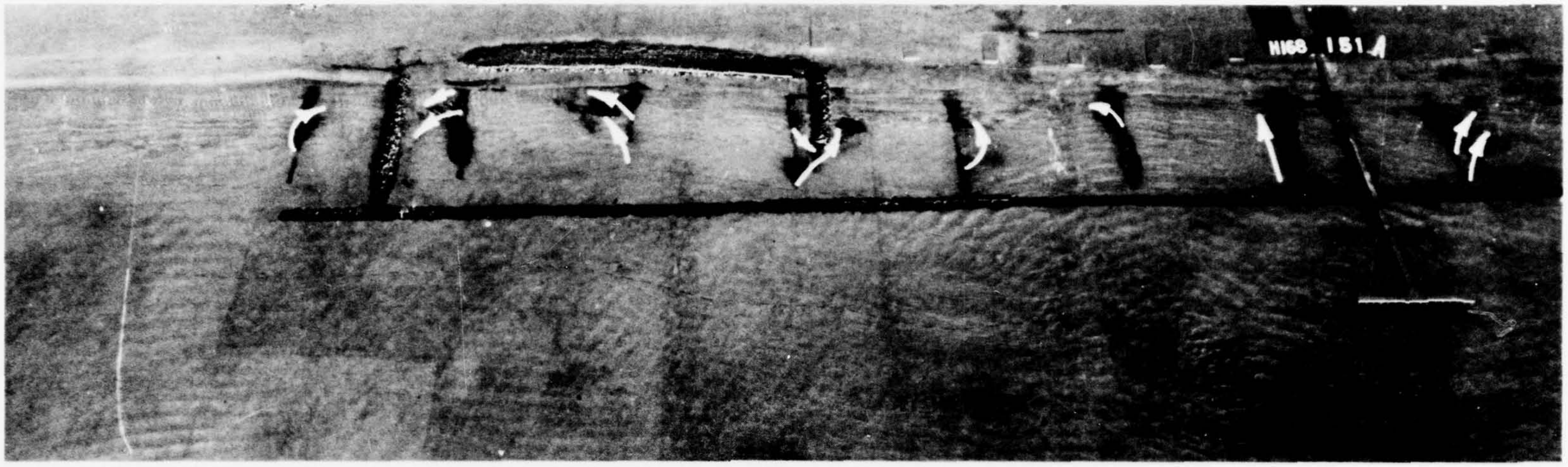


Photo 55. Typical tracer movement for Plan 4 resulting from 10-sec, 9-ft waves from N60°W at mhhw



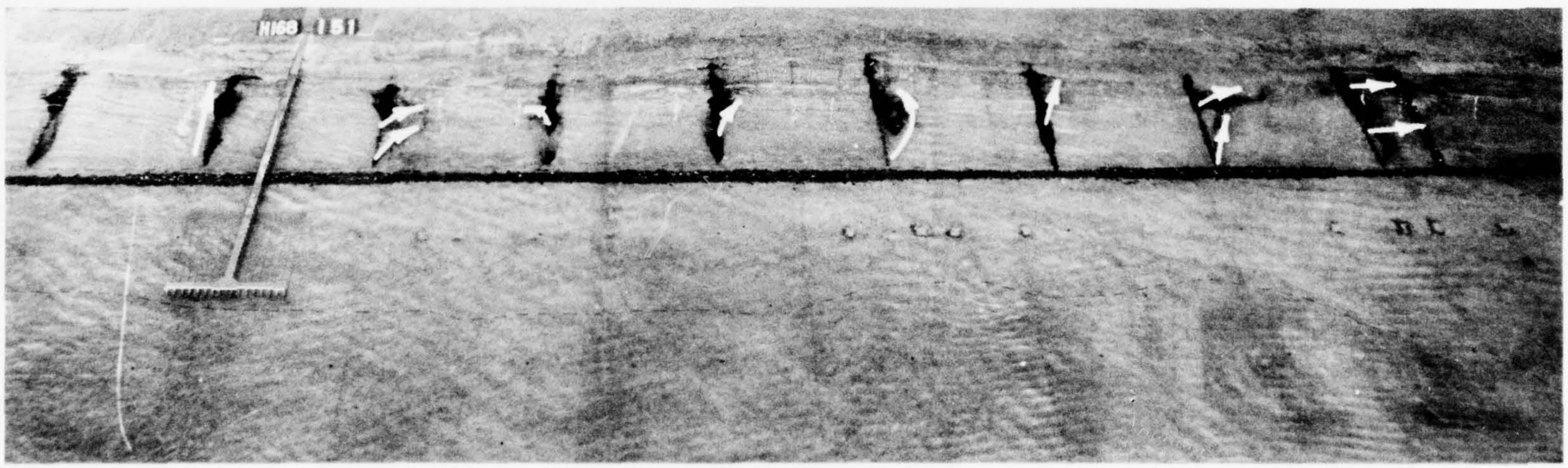


Photo 56. Typical tracer movement for Plan 4 resulting from 7-sec, 11-ft waves from west at mllw

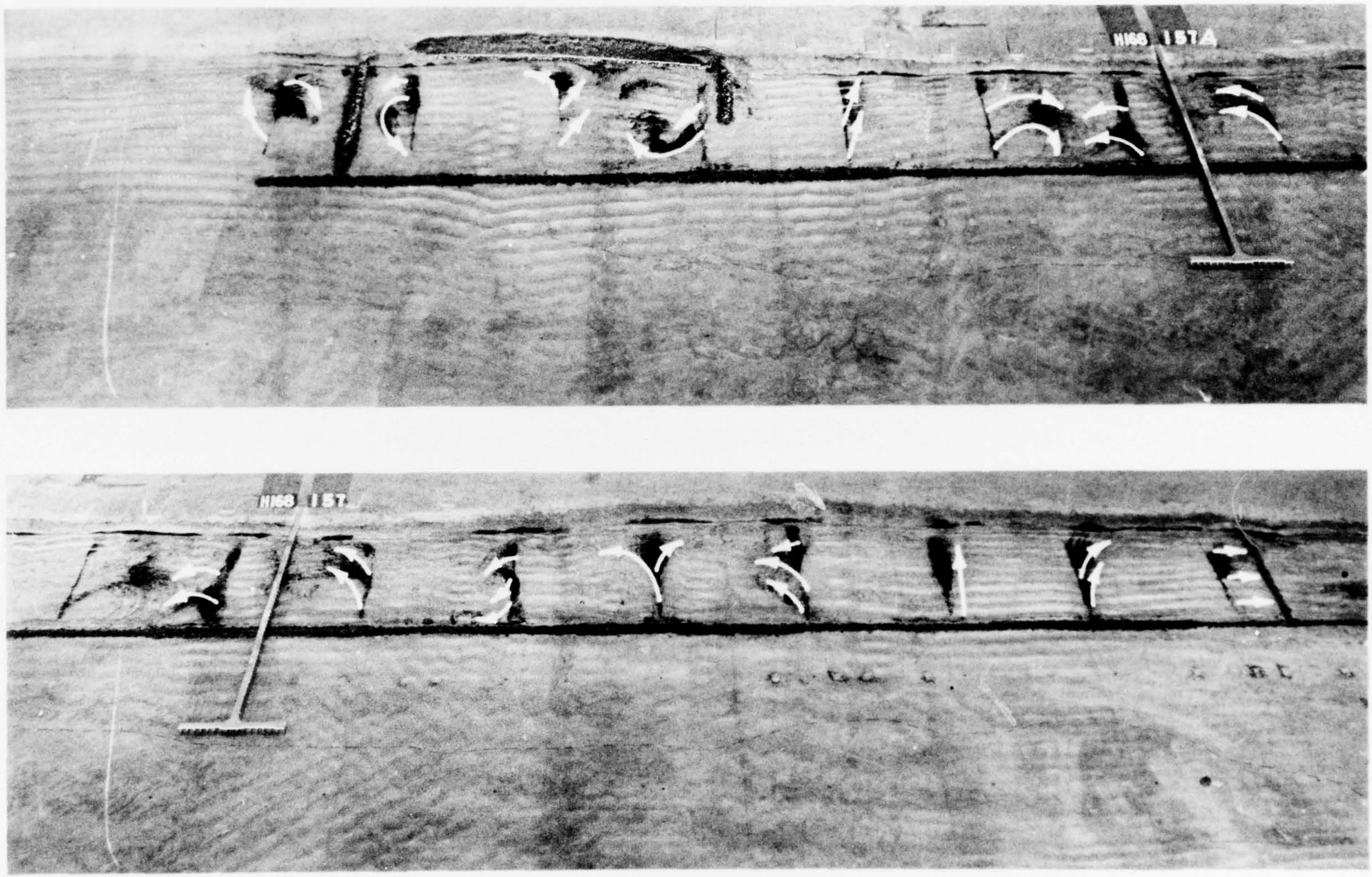


Photo 57. Typical tracer movement for Plan 4 resulting from 7-sec, 11-ft waves from west at mhhw

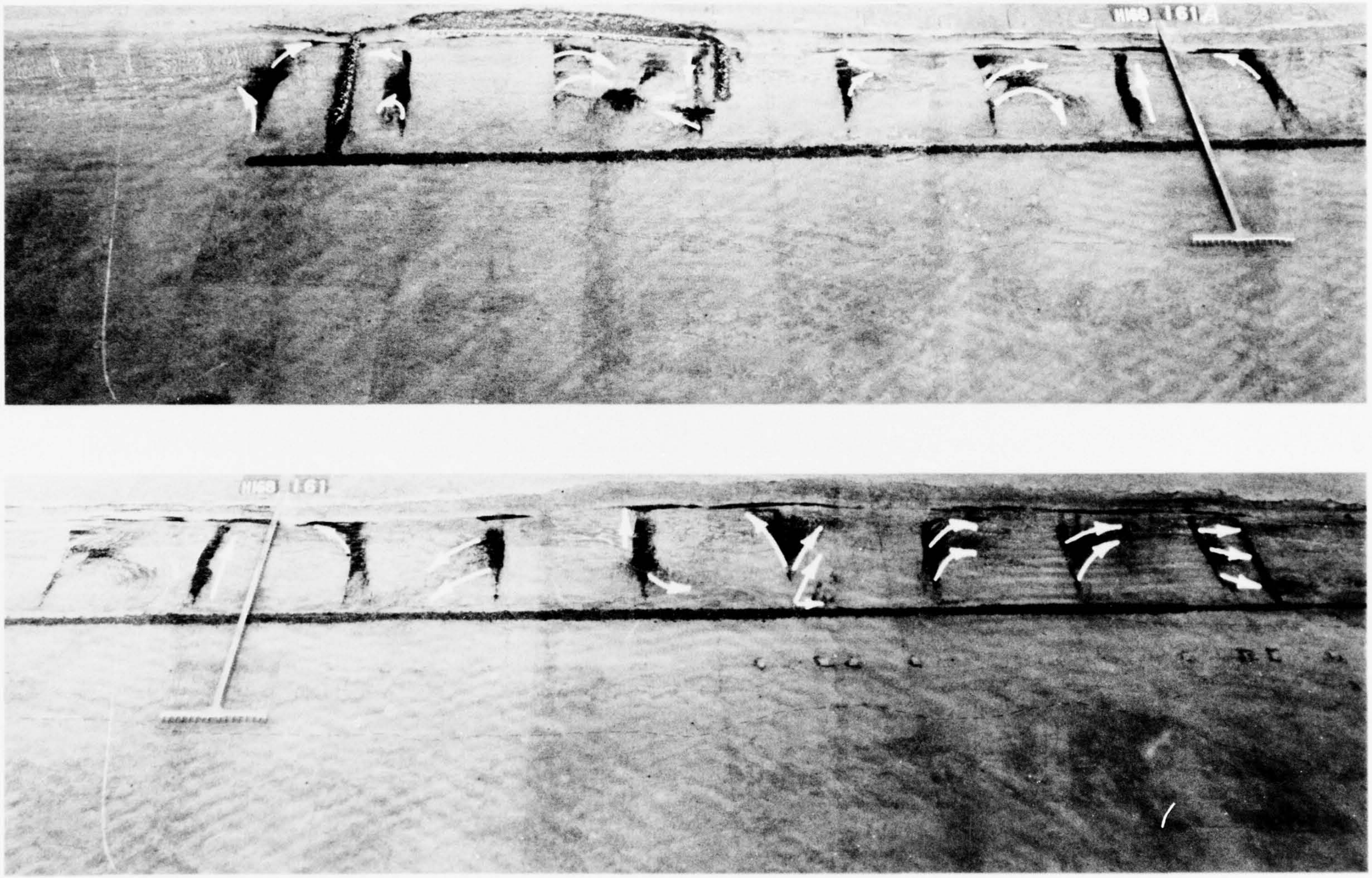


Photo 58. Typical tracer movement for Plan 4 resulting from 14-sec, 9-ft waves from west at mhhw

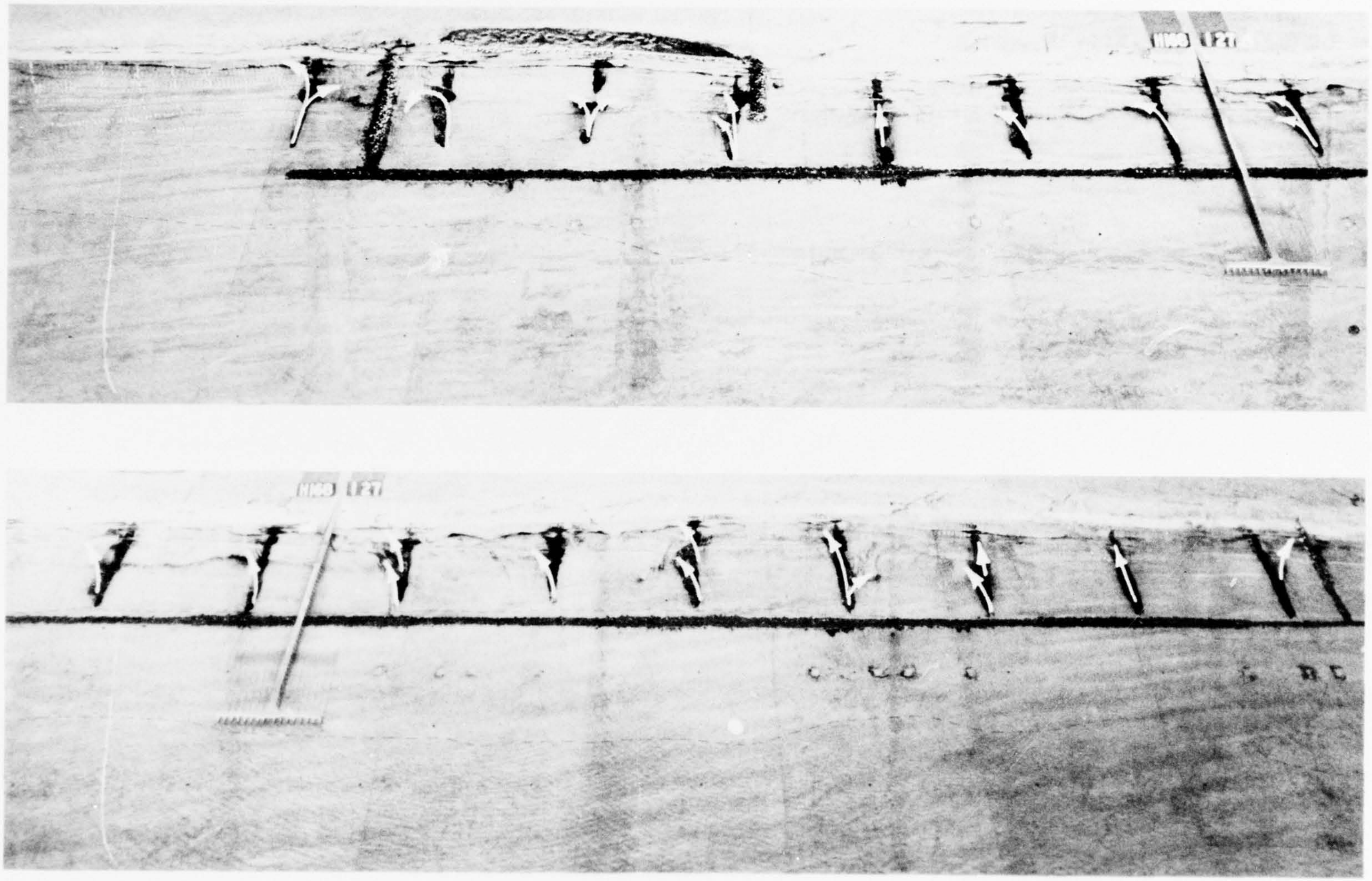
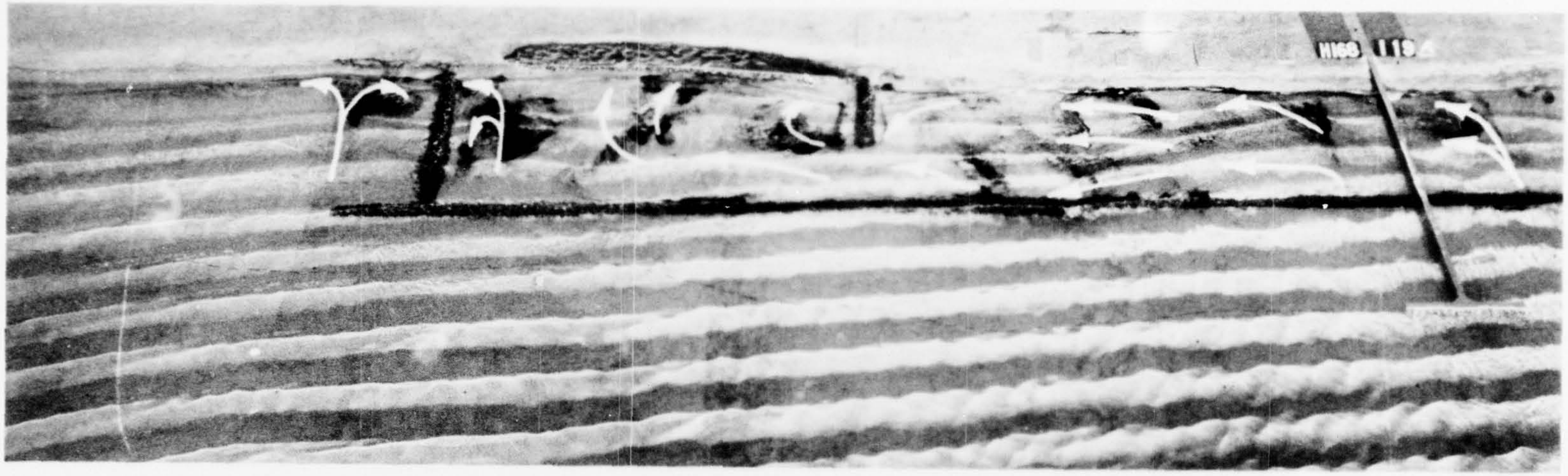


Photo 59. Typical tracer movement for Plan 4 resulting from 7-sec, 11-ft waves from S60°W at mllw



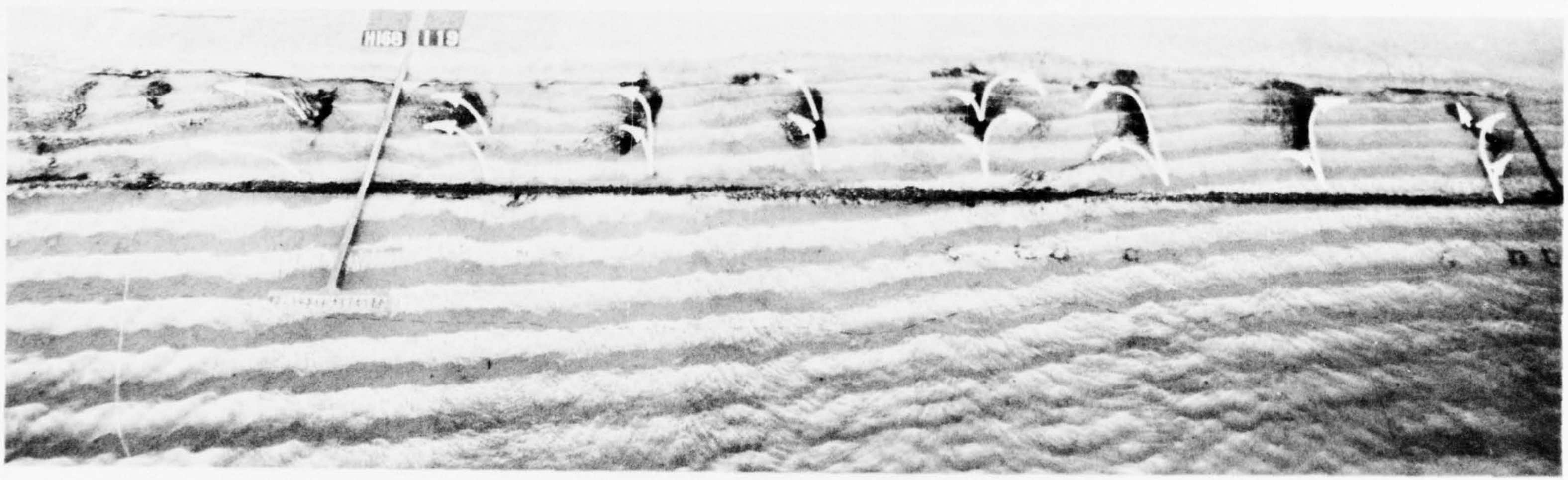


Photo 60. Typical tracer movement for Plan 4 resulting from 7-sec, 11-ft waves from S60°W at mhhw

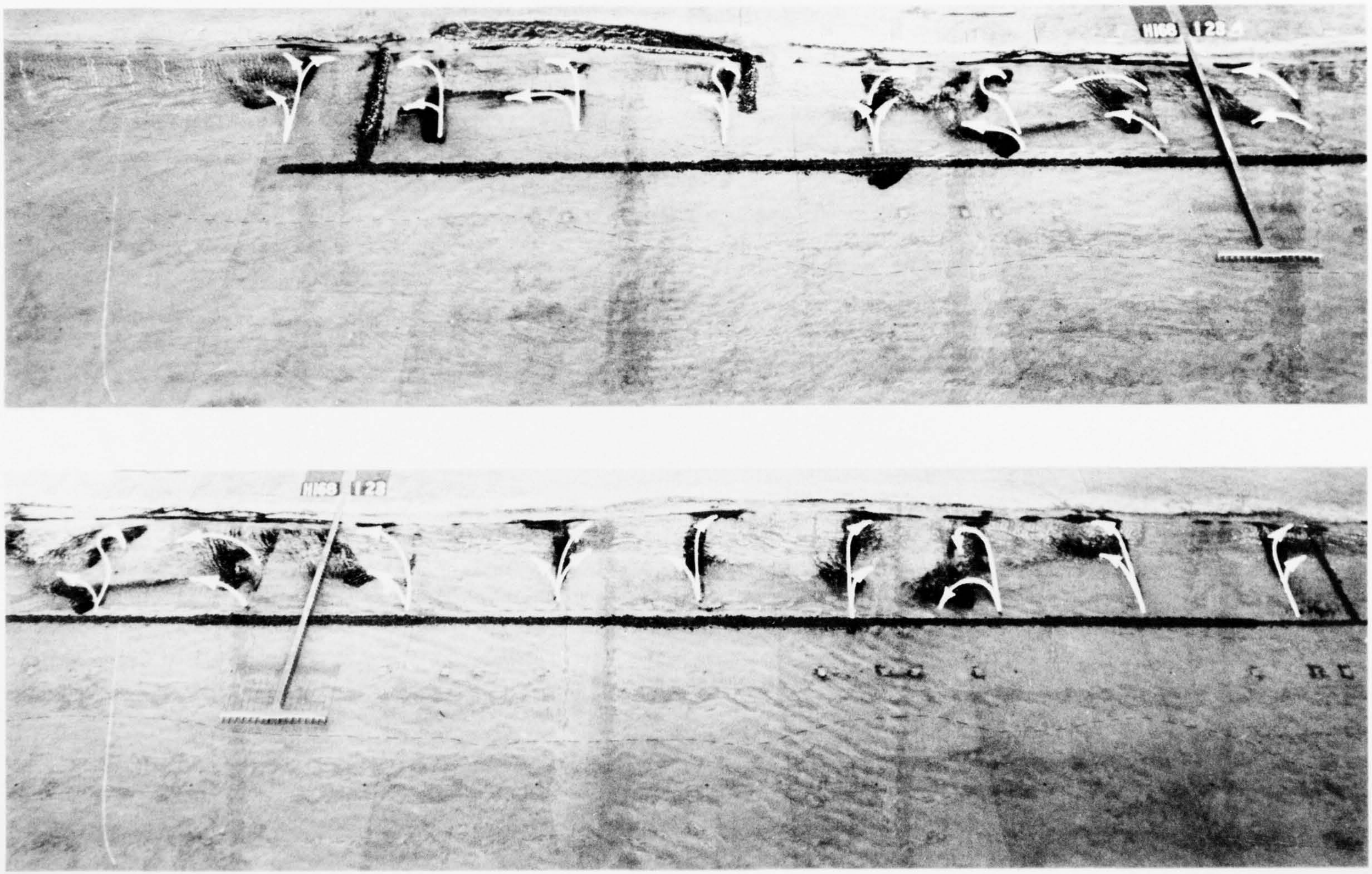


Photo 61. Typical tracer movement for Plan 4 resulting from 14-sec, 7-ft waves from S60°W at mhhw

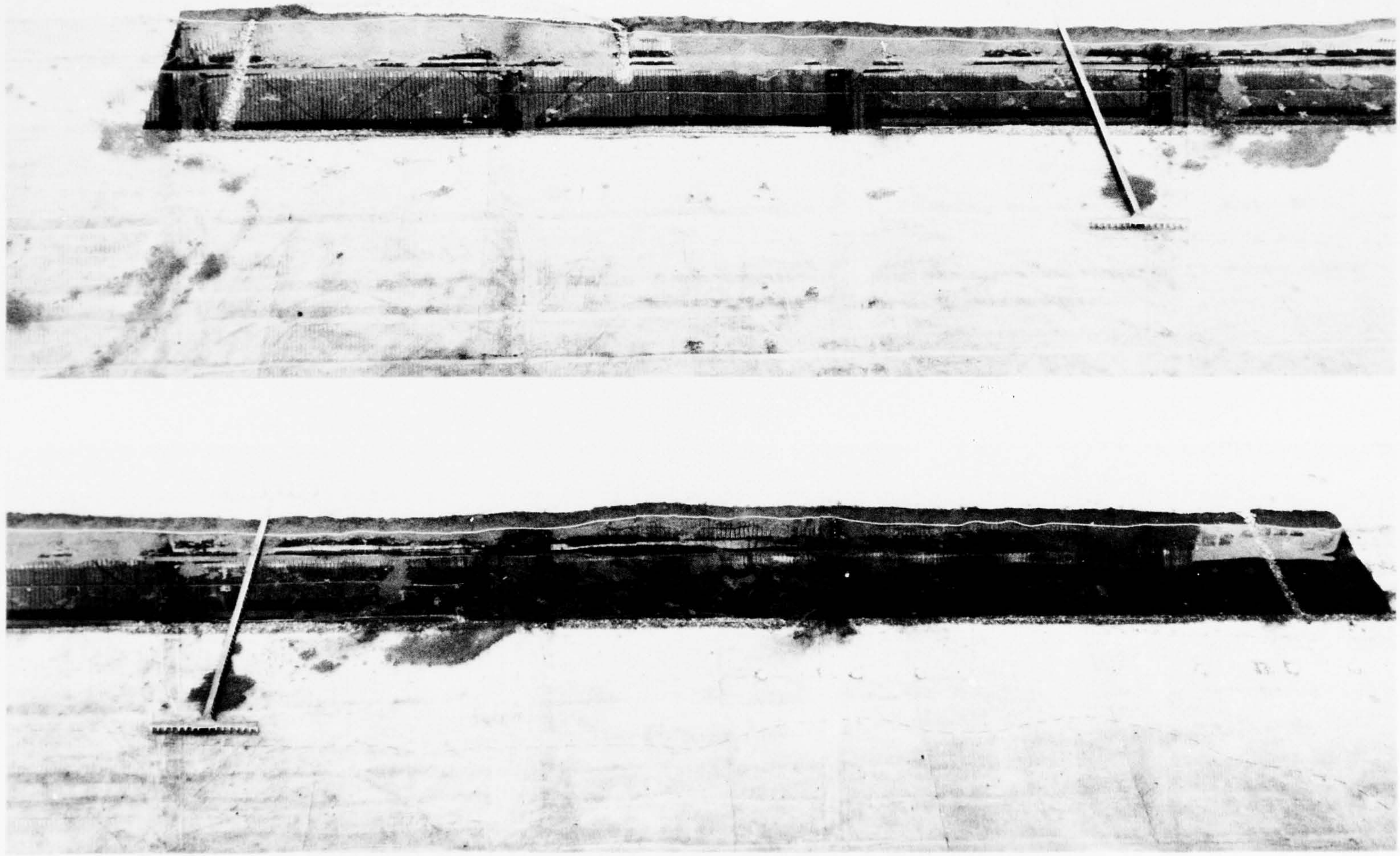


Photo 62. Semimovable-bed tracer placement for Plan 4

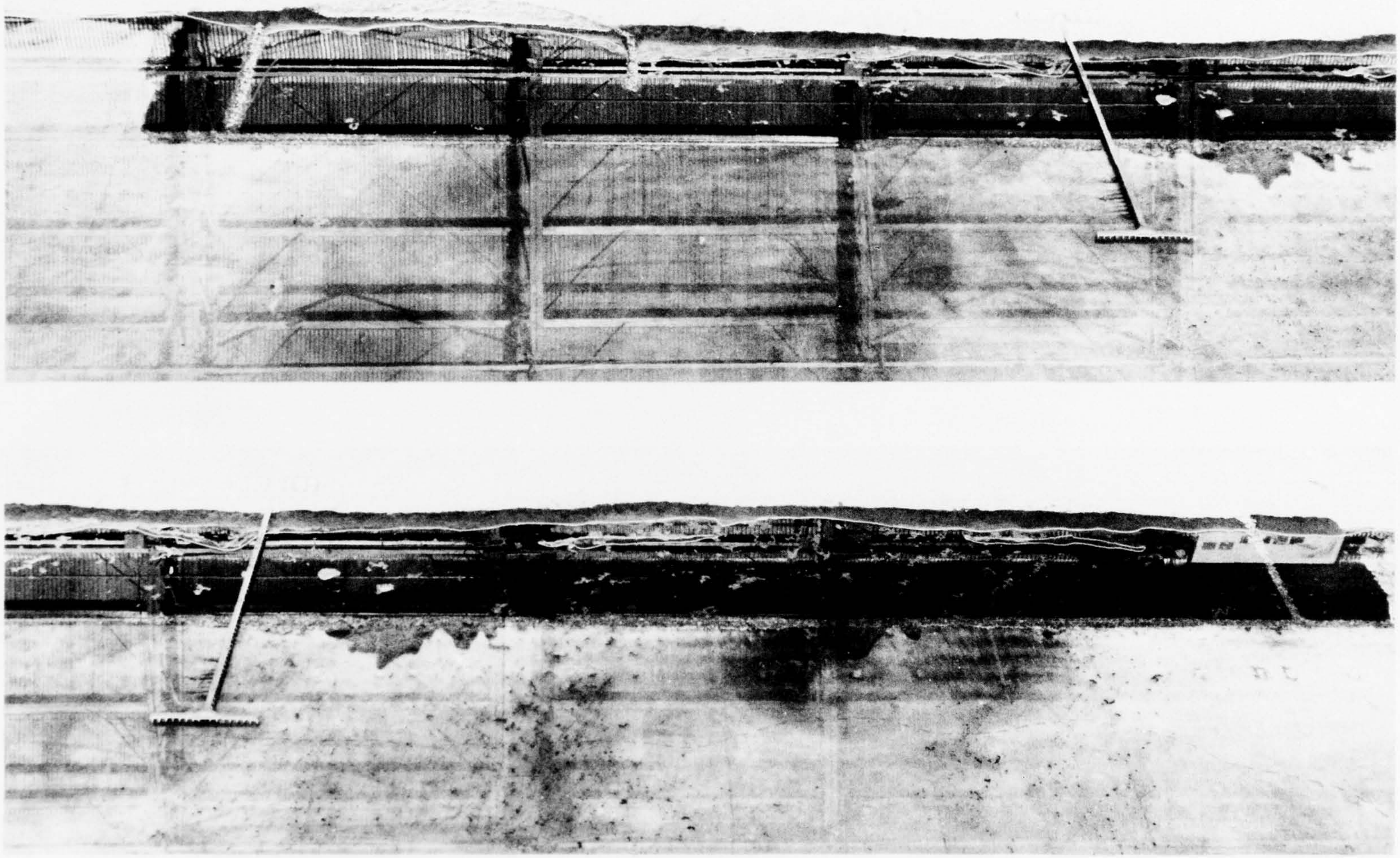


Photo 63. Semimovable-bed tracer deposits for Plan 4 resulting from 7-sec, 4-ft waves from S60°W at mhhw

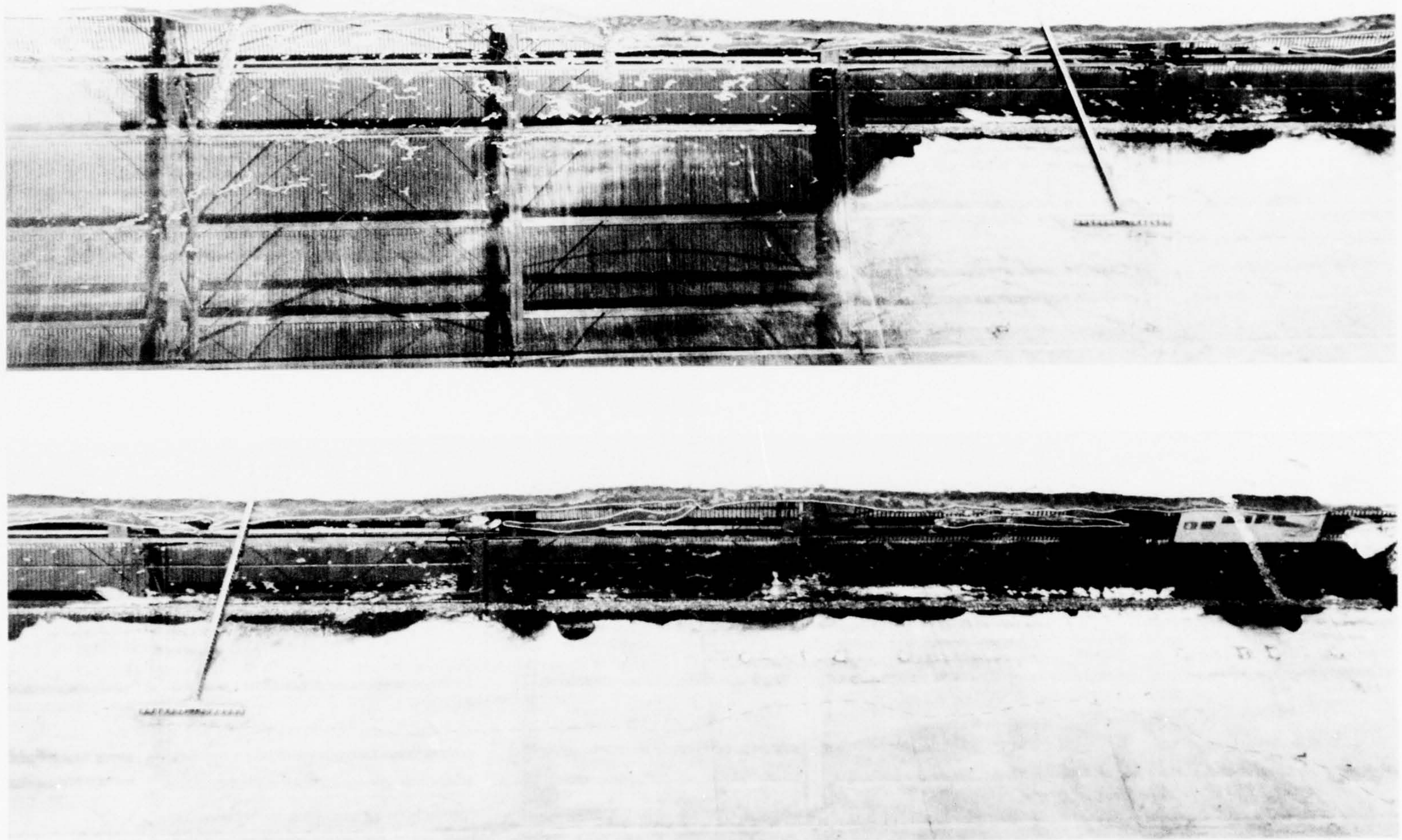


Photo 64. Semimovable-bed tracer deposits for Plan 4 resulting from 7-sec, 11-ft waves from S60°W at mhhw

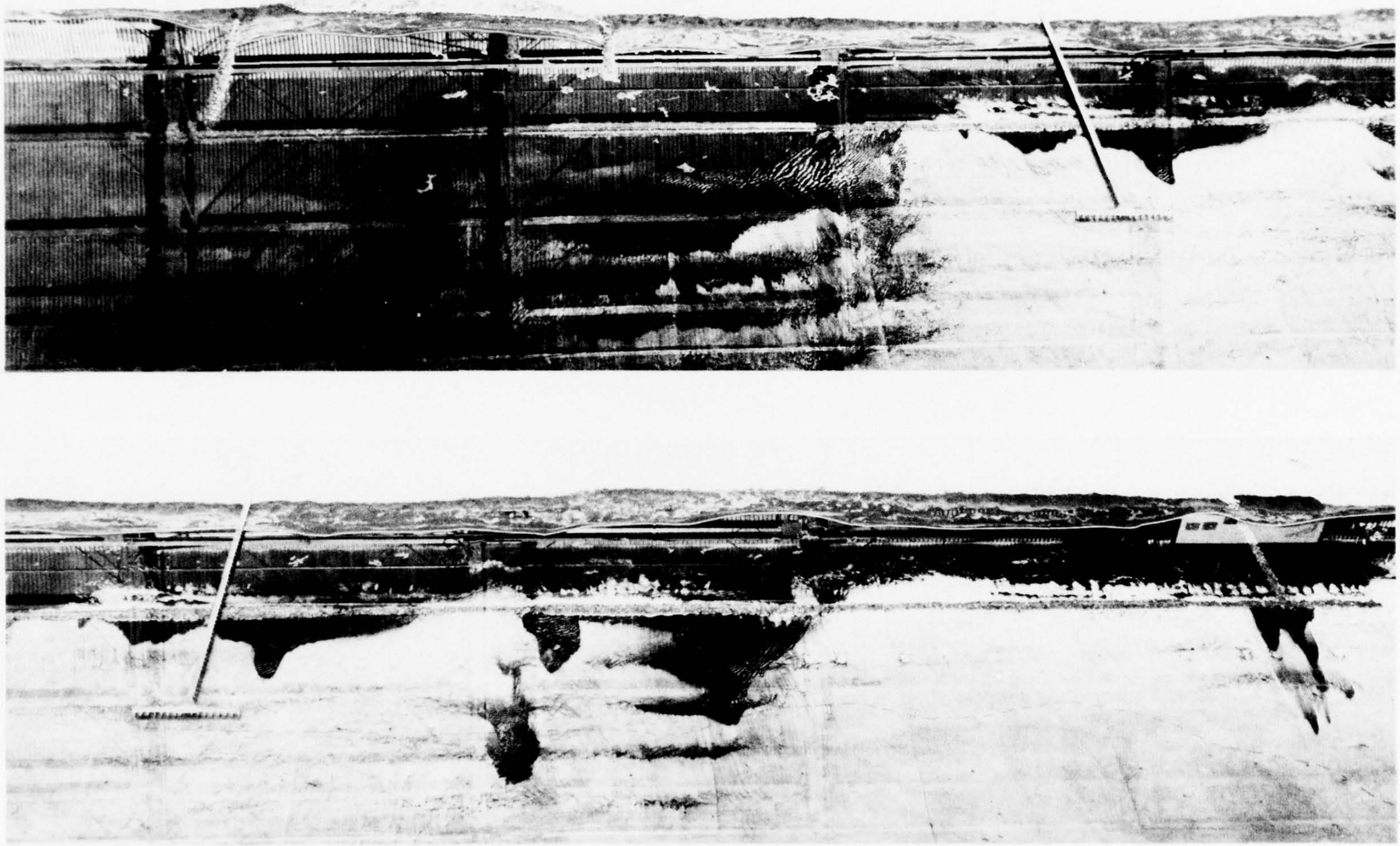
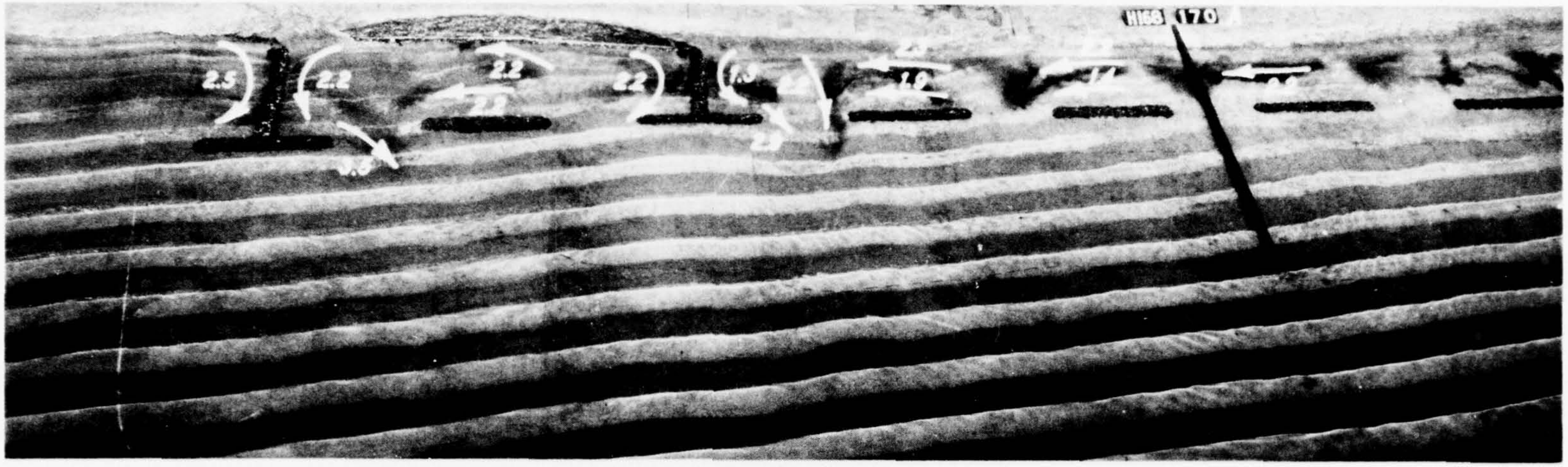


Photo 65. Semimovable-bed tracer deposits for Plan 4 resulting from 14-sec, 7-ft waves from S60°W at mhhw



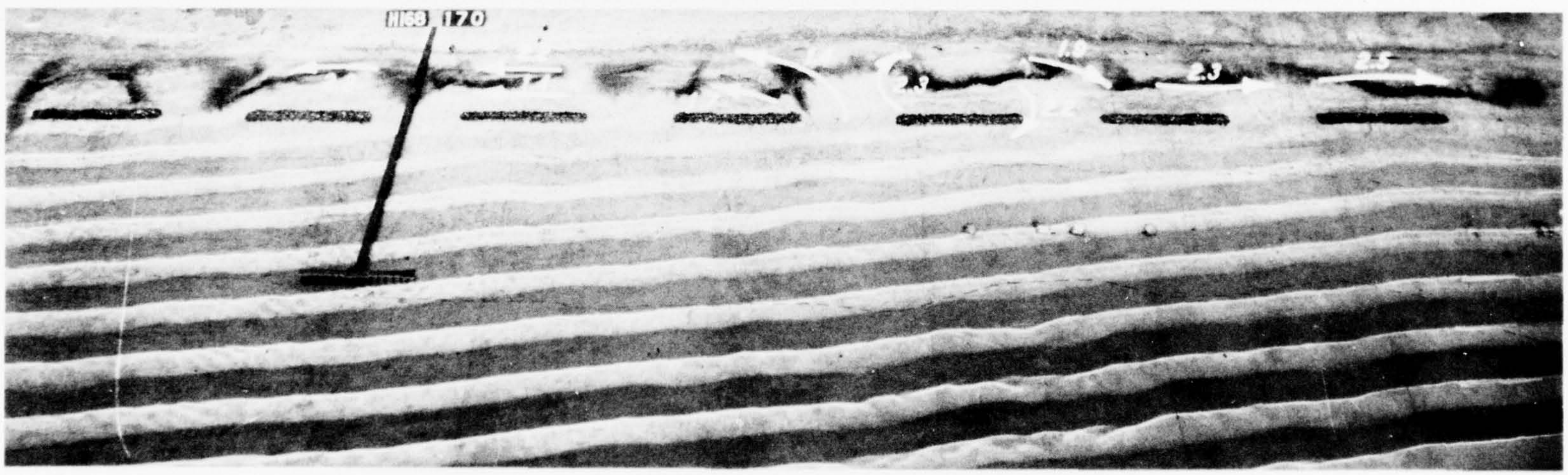


Photo 66. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 5; 7-sec, 11-ft waves from S60°W at mhhw

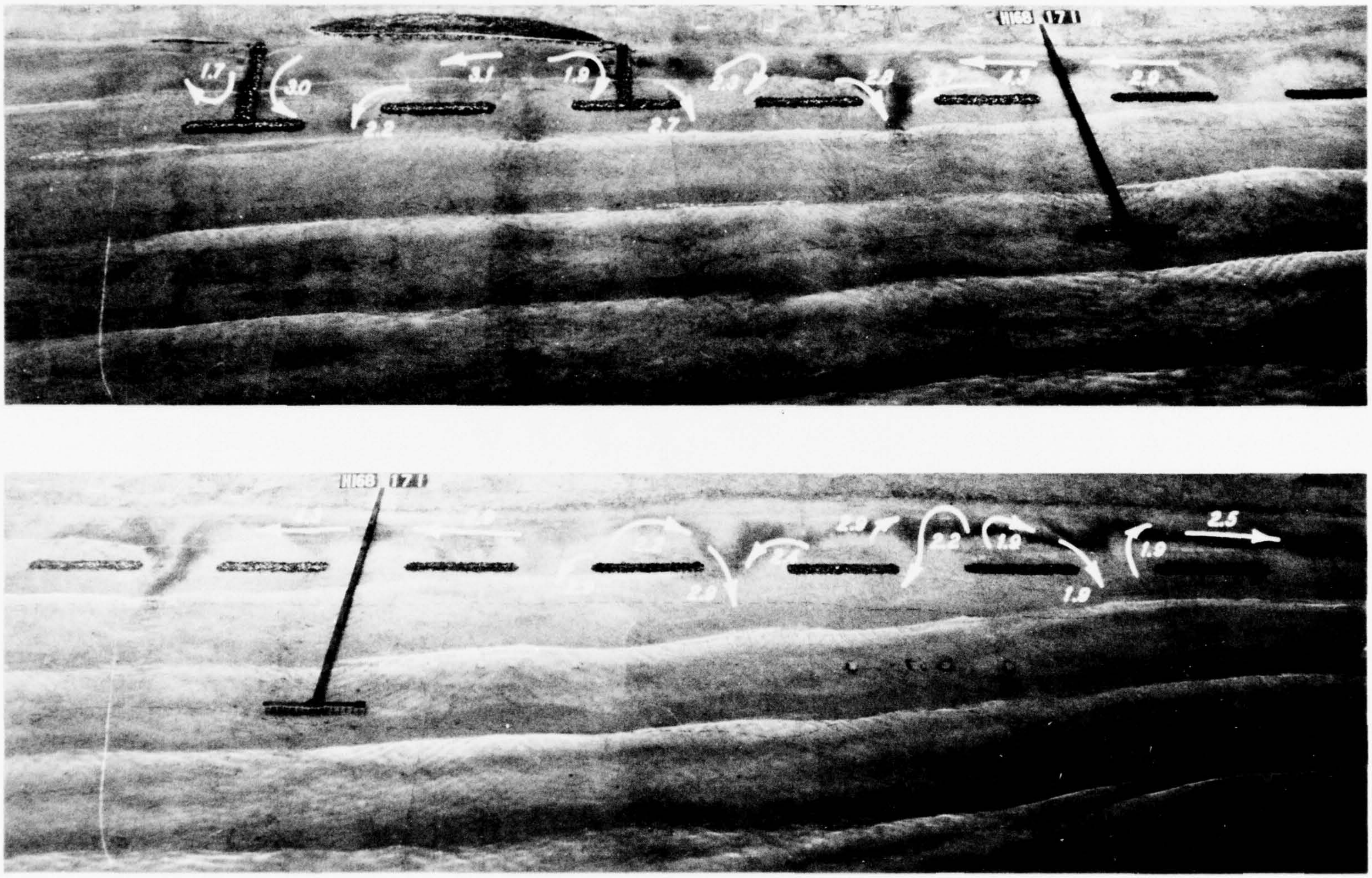


Photo 67. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 5; 14-sec, 7-ft waves from S60°W at mhhw

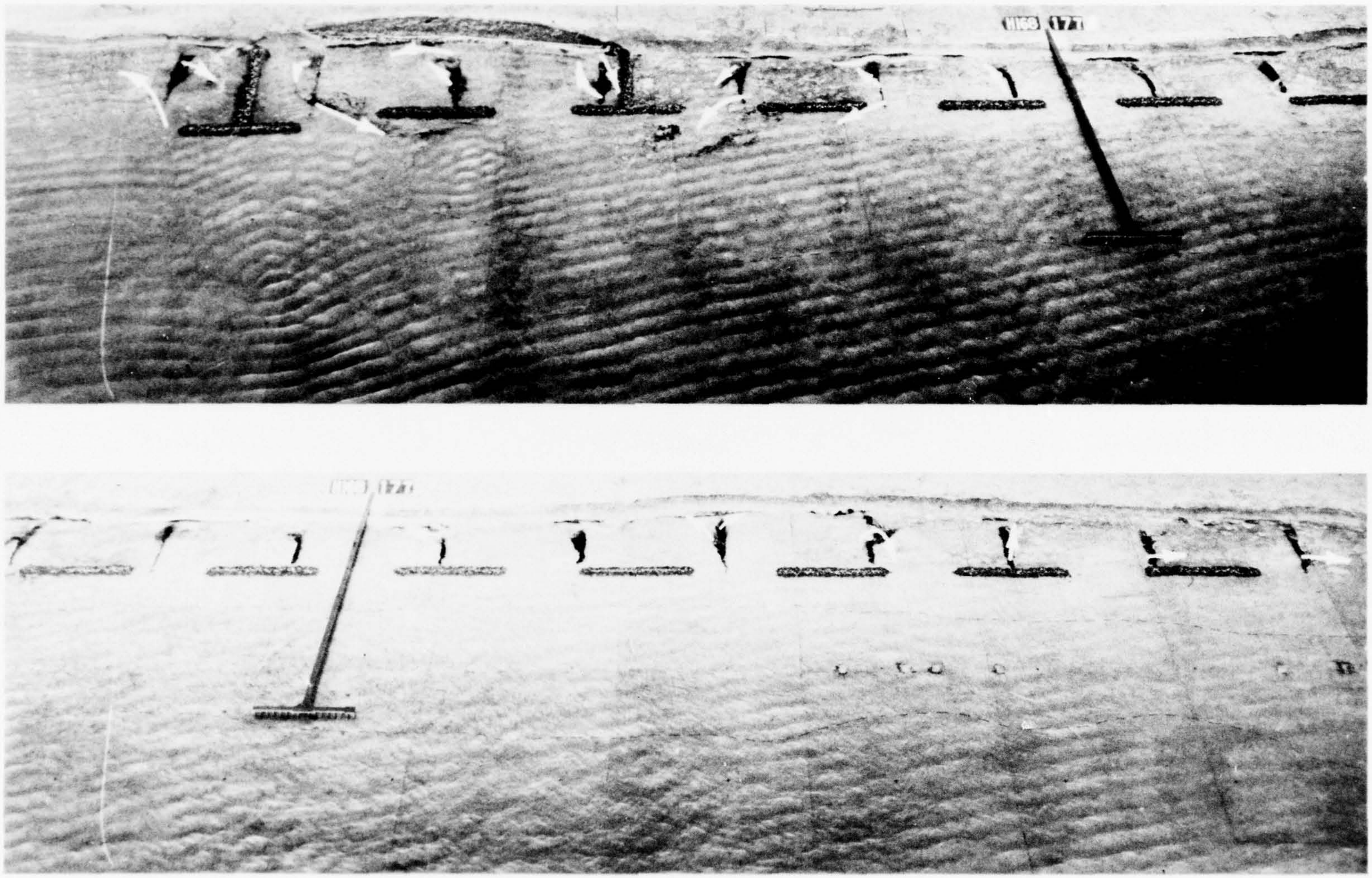


Photo 68. Typical tracer movement for Plan 5 resulting from 7-sec, 11-ft waves from S60°W at mhhw

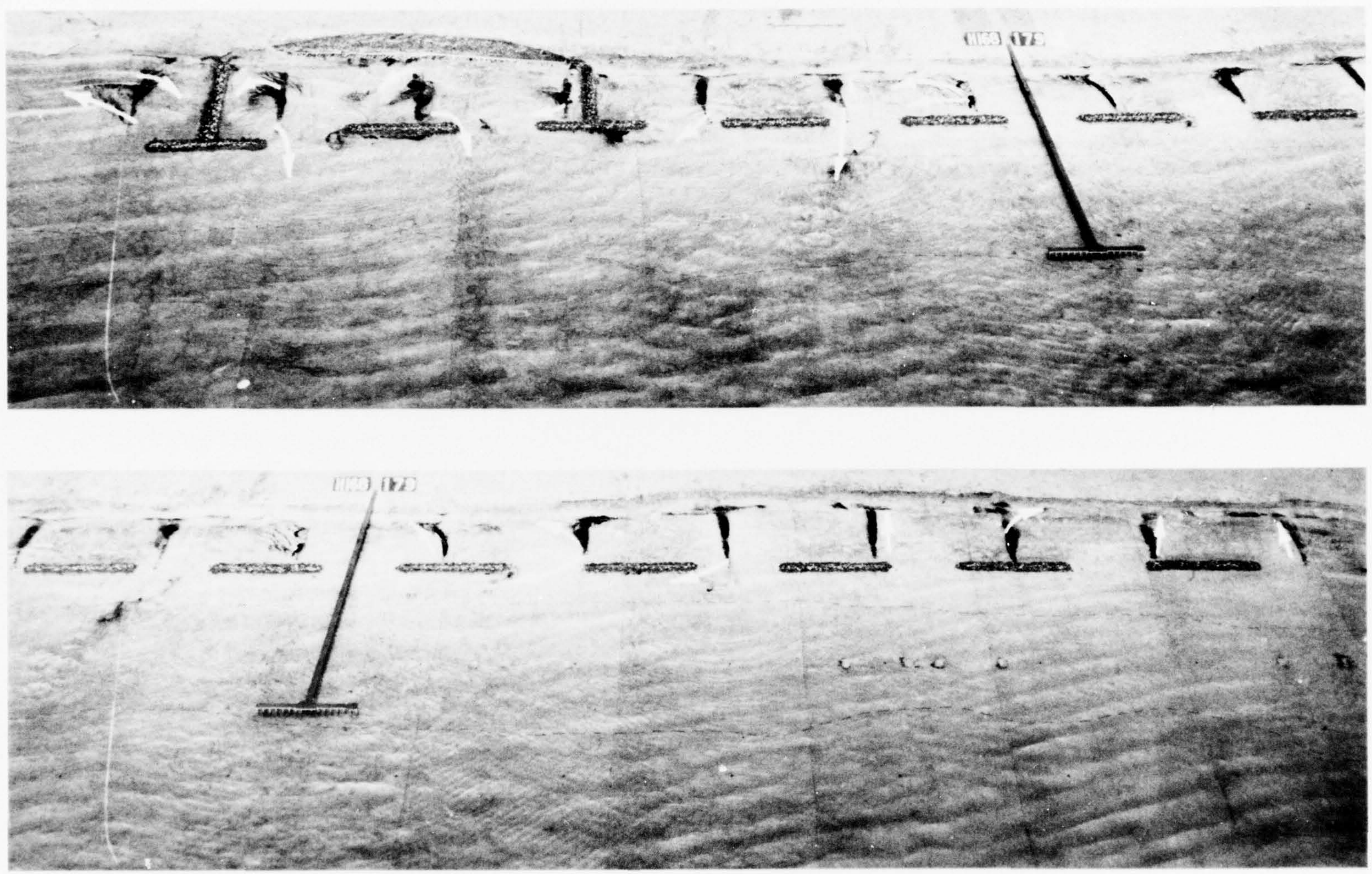
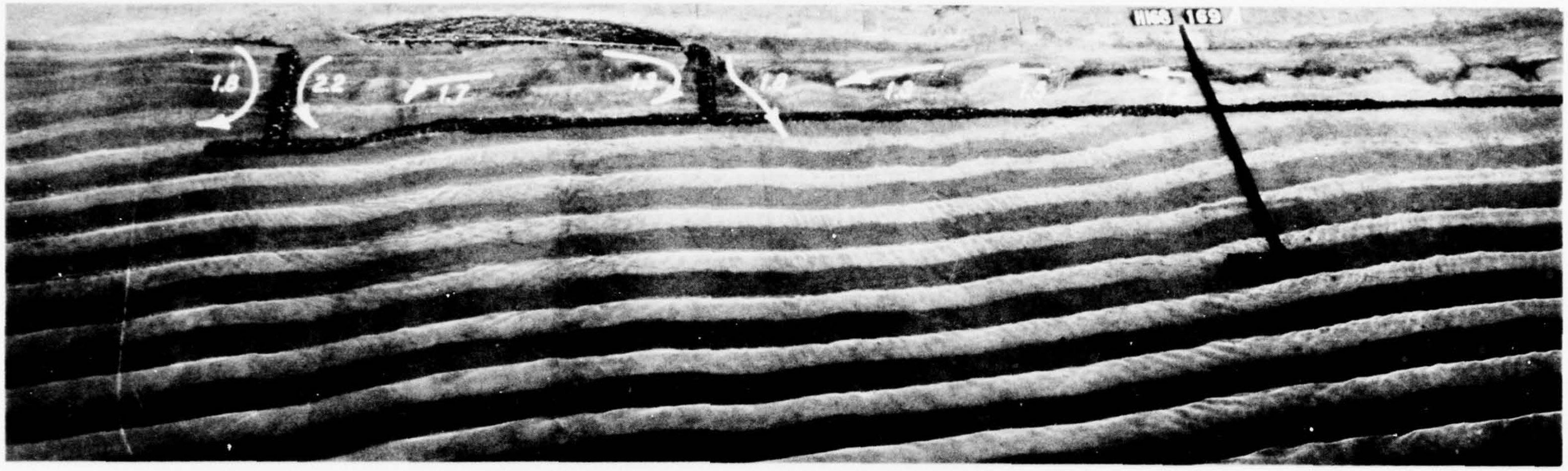


Photo 69. Typical tracer movement for Plan 5 resulting from 14-sec, 7-ft waves from S60°W at mhhw



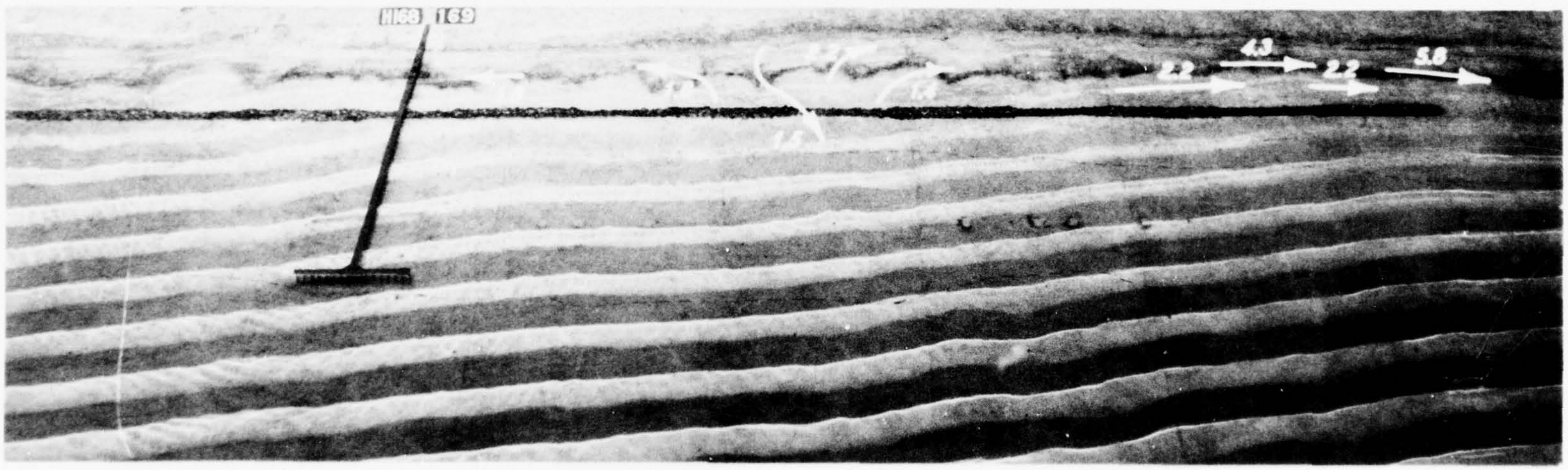


Photo 70. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 5A; 7-sec, 11-ft waves from S60°W at mhhw

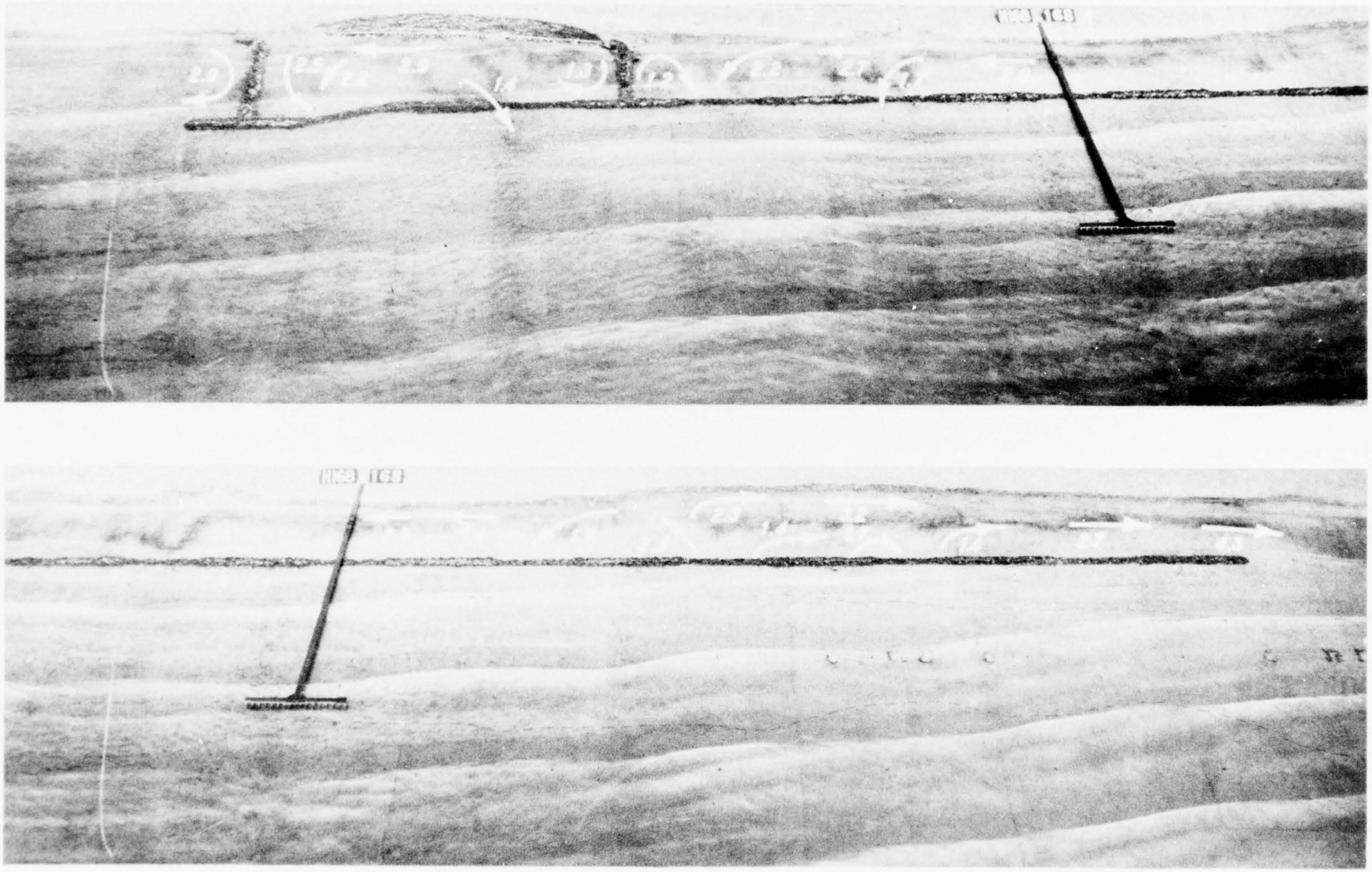


Photo 71. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 5A; 14-sec, 7-ft waves from S60°W at mhhw

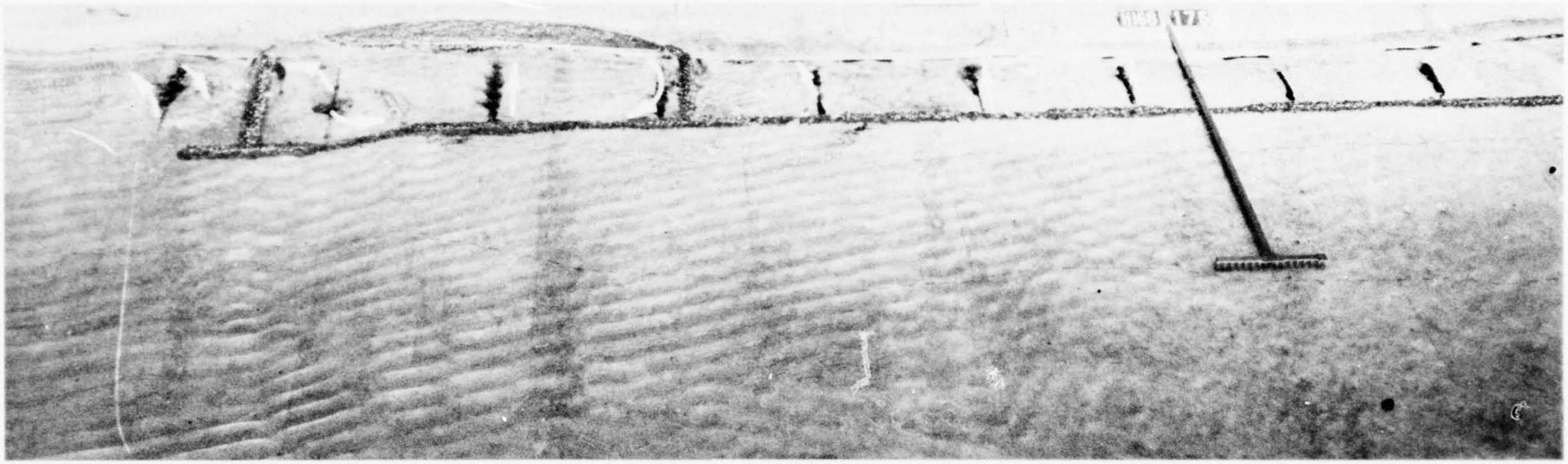


Photo 72. Typical tracer movement for Plan 5A resulting from 7-sec, 11-ft waves from S60°W at mhhw

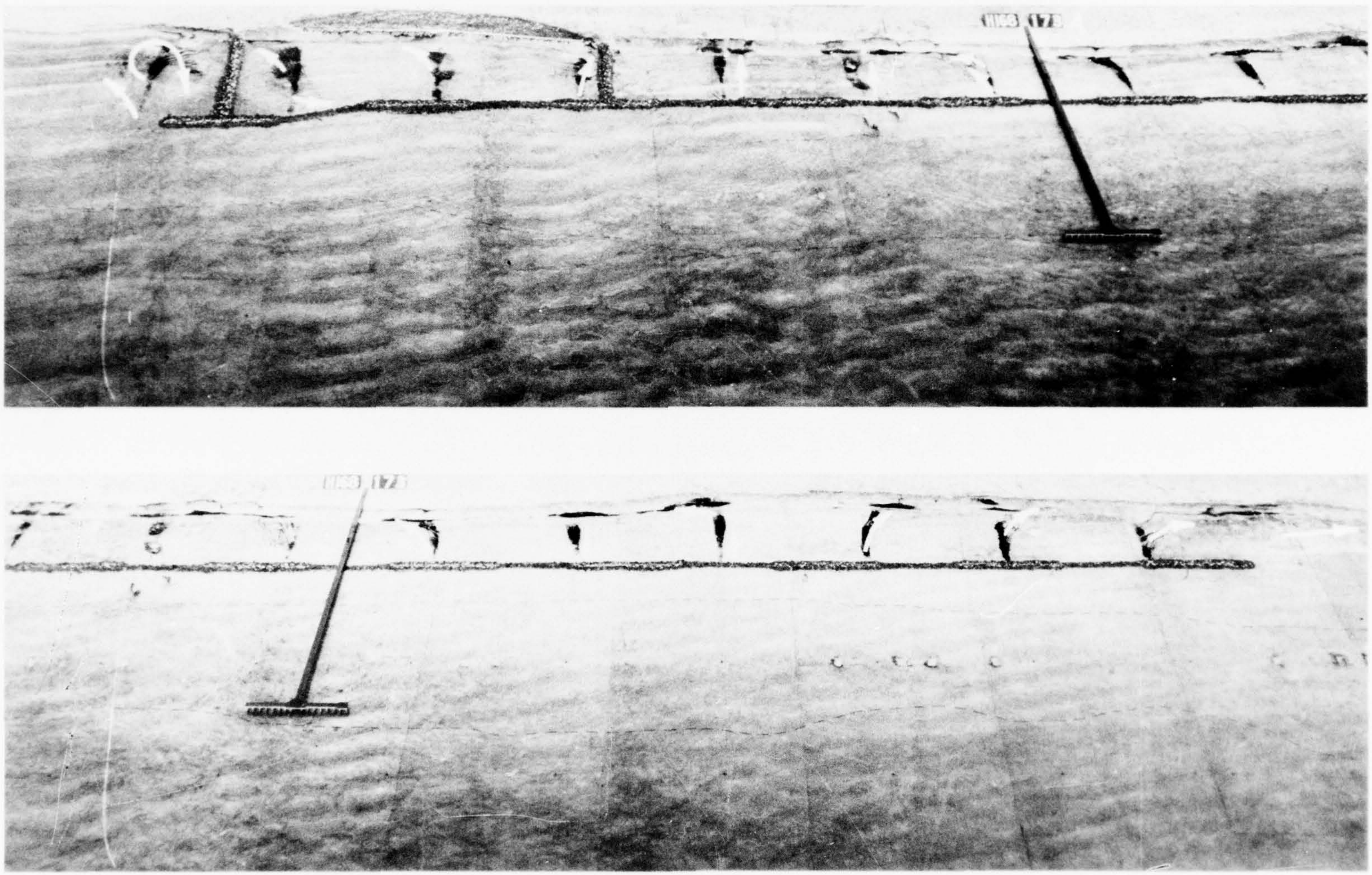


Photo 73. Typical tracer movement for Plan 5A resulting from 14-sec, 7-ft waves from S60°W at mhhw

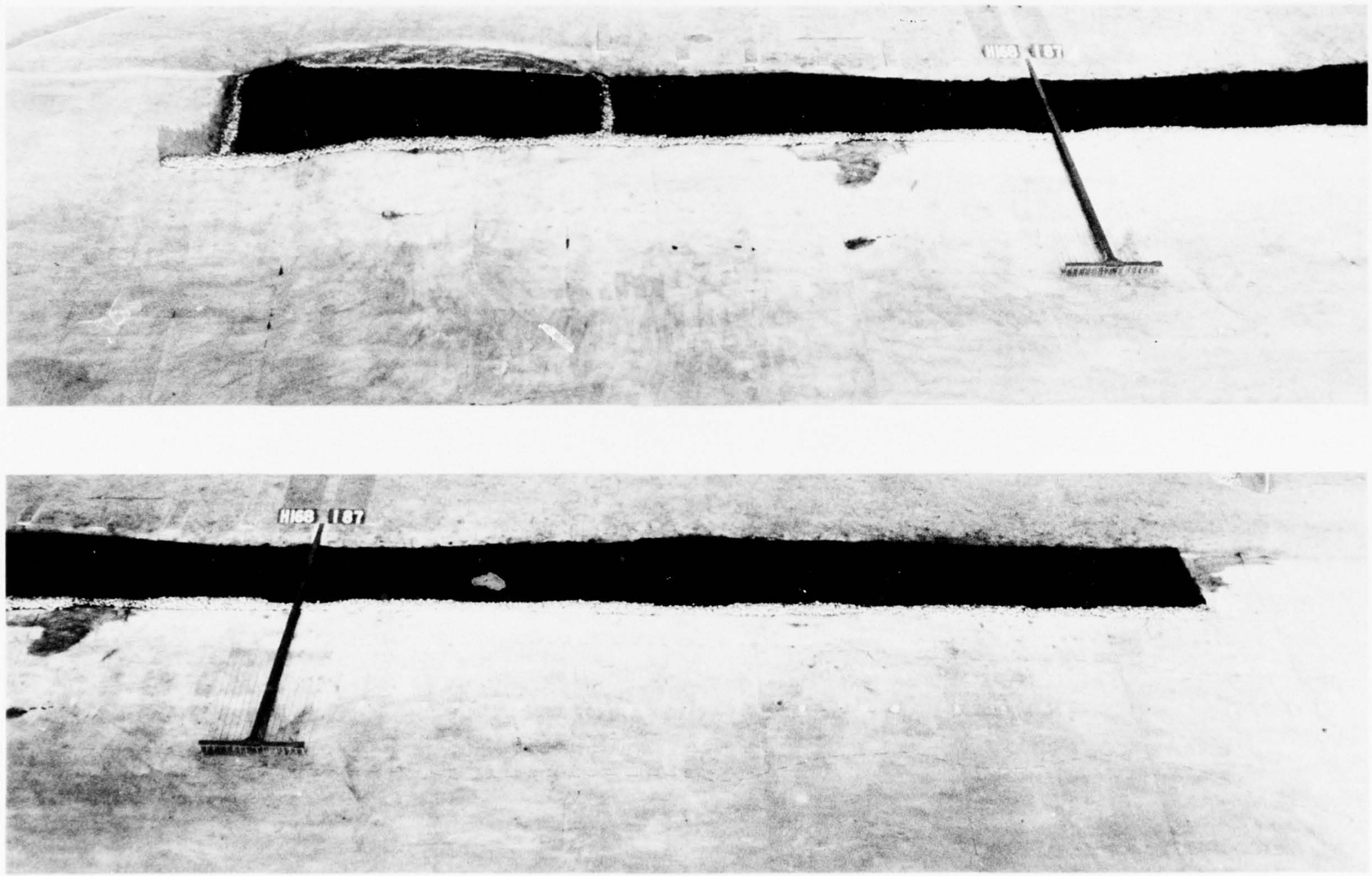


Photo 74. Semimovable-bed tracer placement for Plan 5A

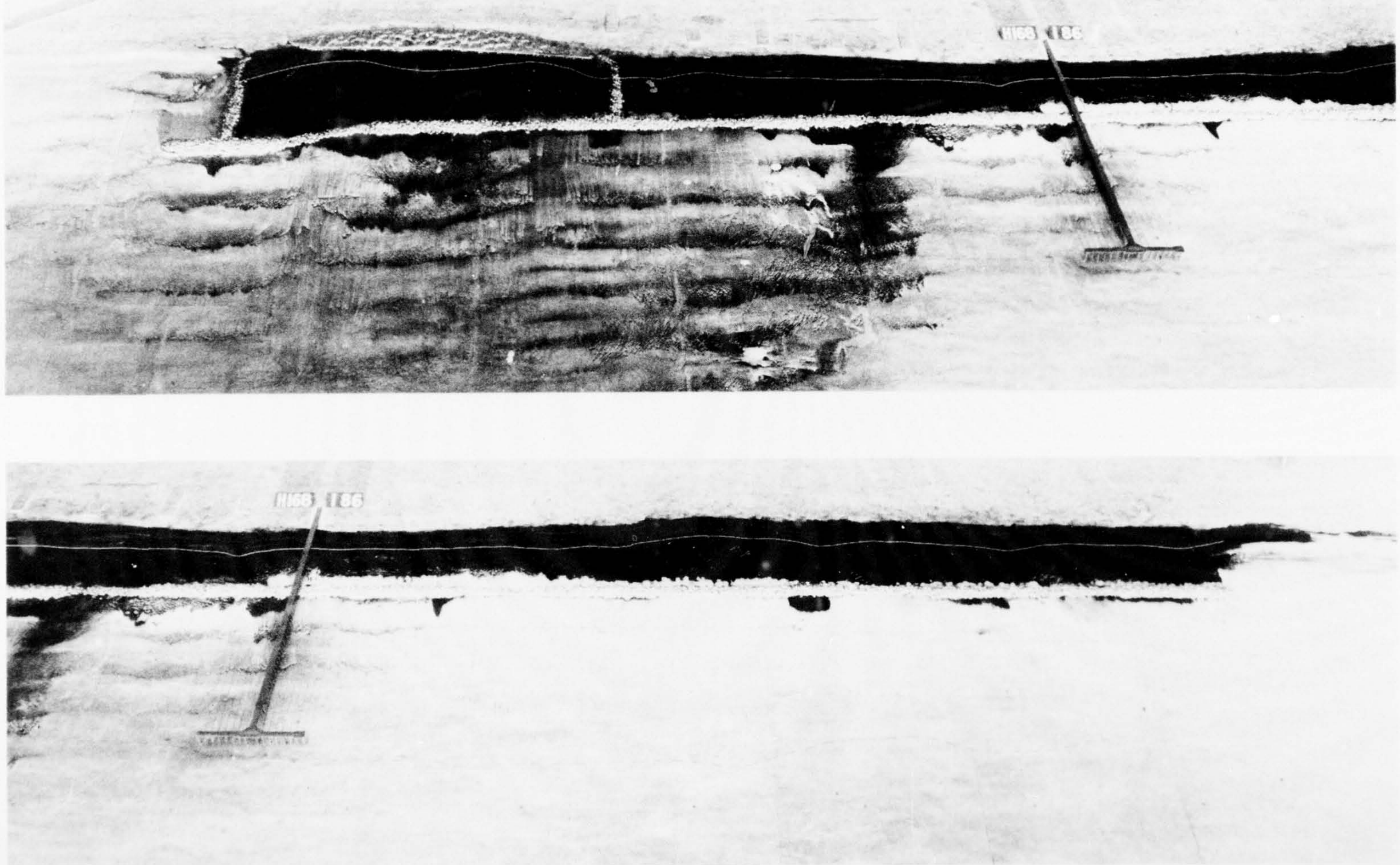


Photo 75. Semimovable-bed tracer deposits for Plan 5A resulting from 7-sec, 4- and 11-ft waves and 14-sec, 4- and 7-ft waves from S60°W at mhhw

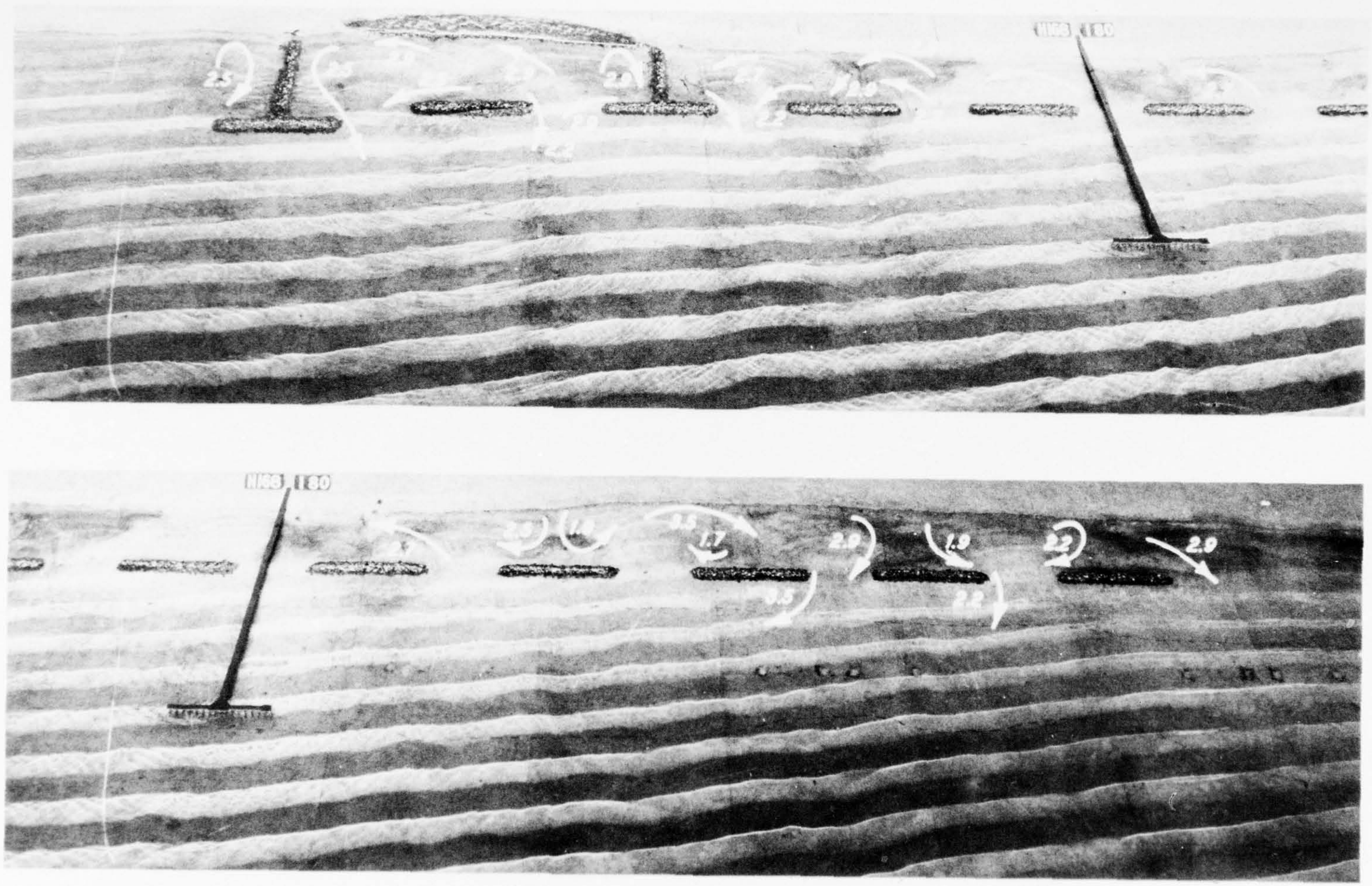
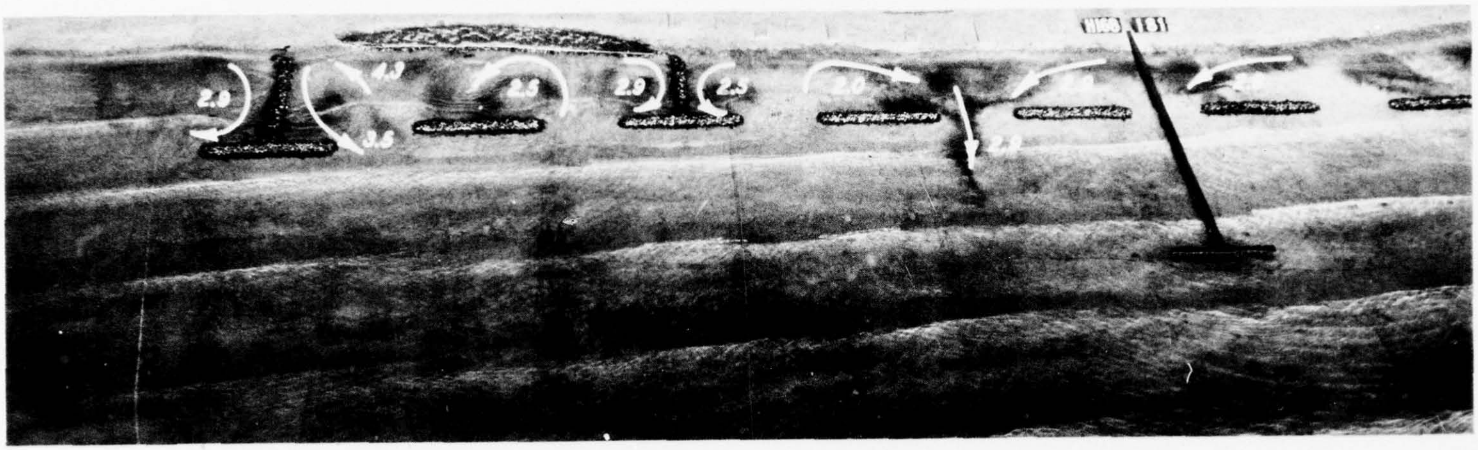


Photo 76. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 5B; 7-sec, 11-ft waves from S60°W at mhhw



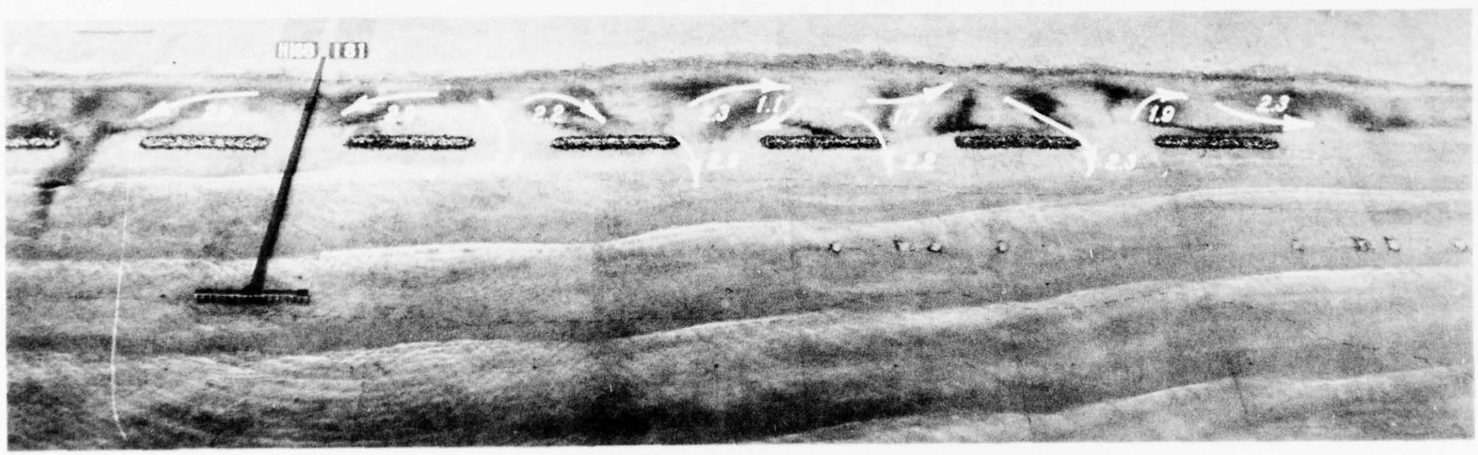


Photo 77. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 5B; 14-sec, 7-ft waves from S60°W at mhhw

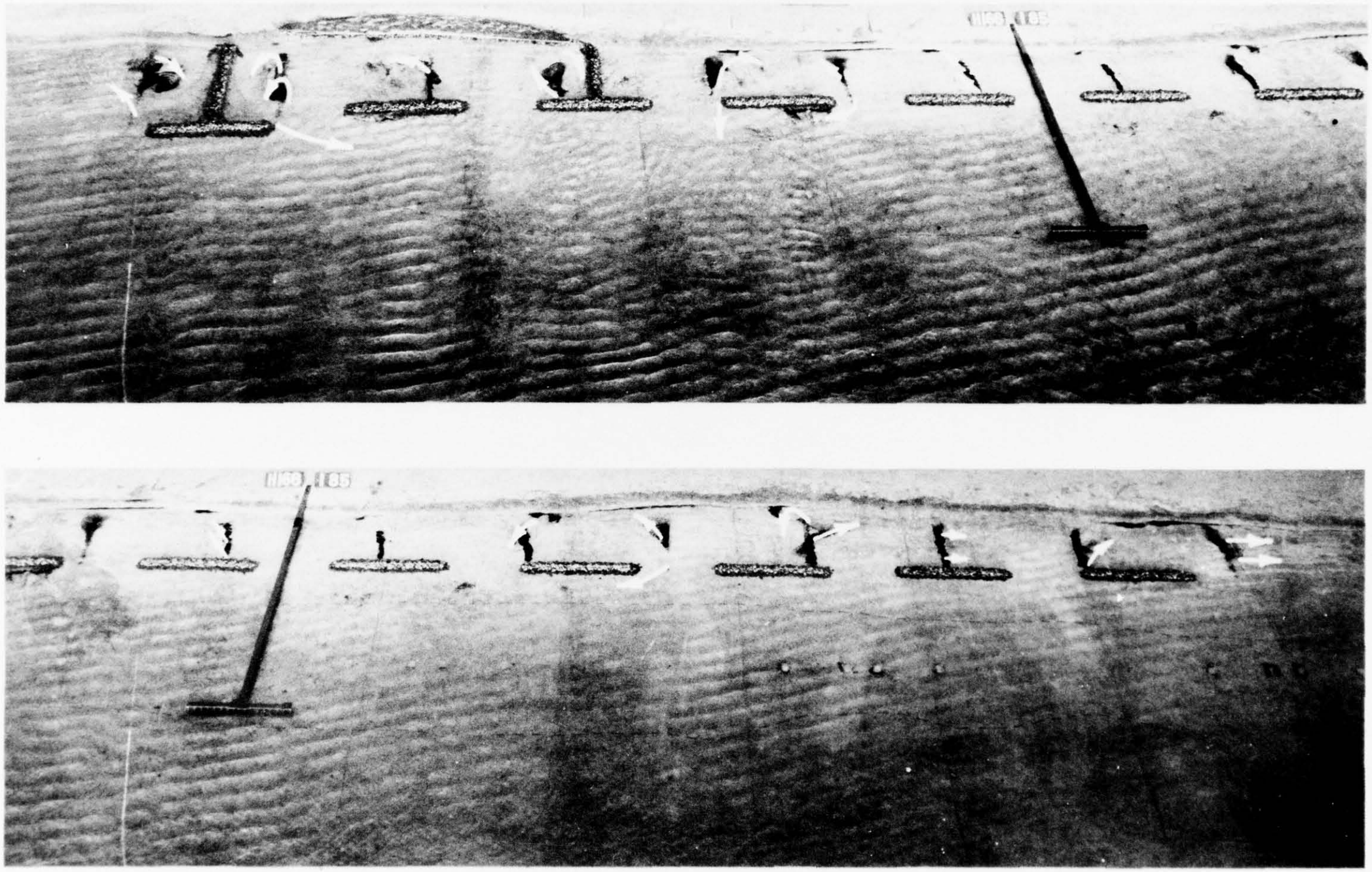


Photo 78. Typical tracer movement for Plan 5B resulting from 7-sec, 11-ft waves from S60°W at mhhw

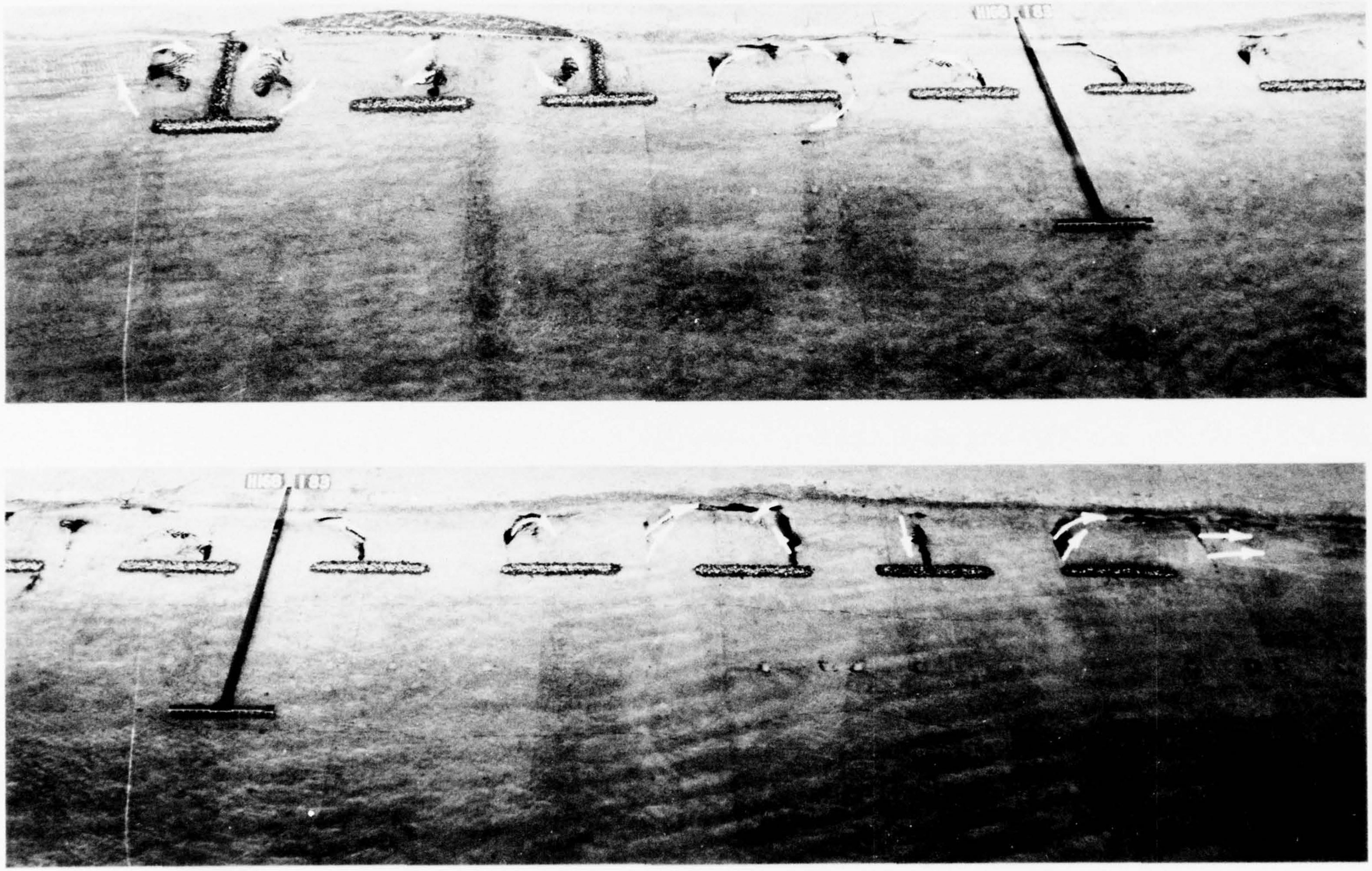


Photo 79. Typical tracer movement for Plan 5B resulting from 14-sec, 7-ft waves from S60°W at mhhw

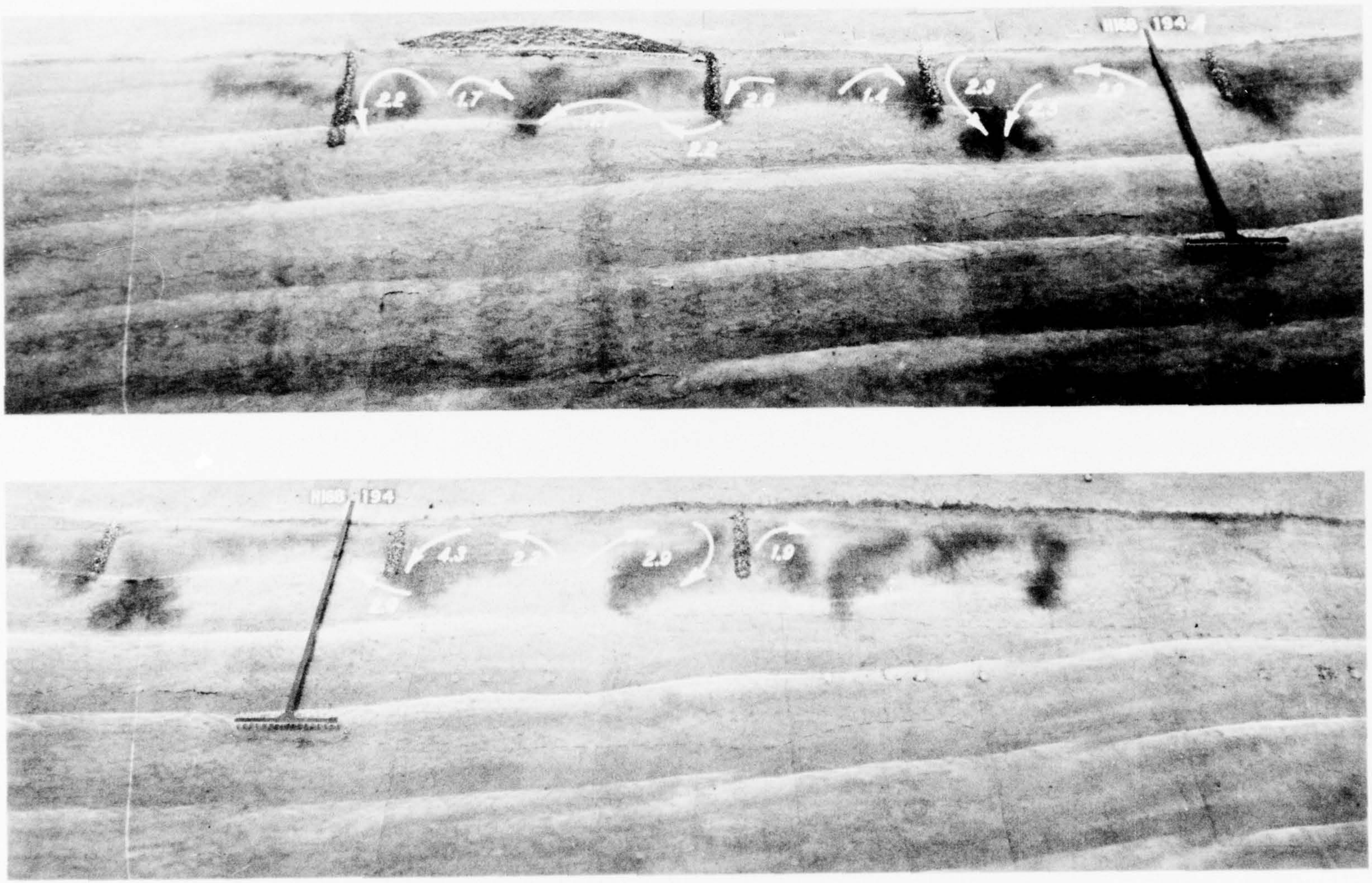
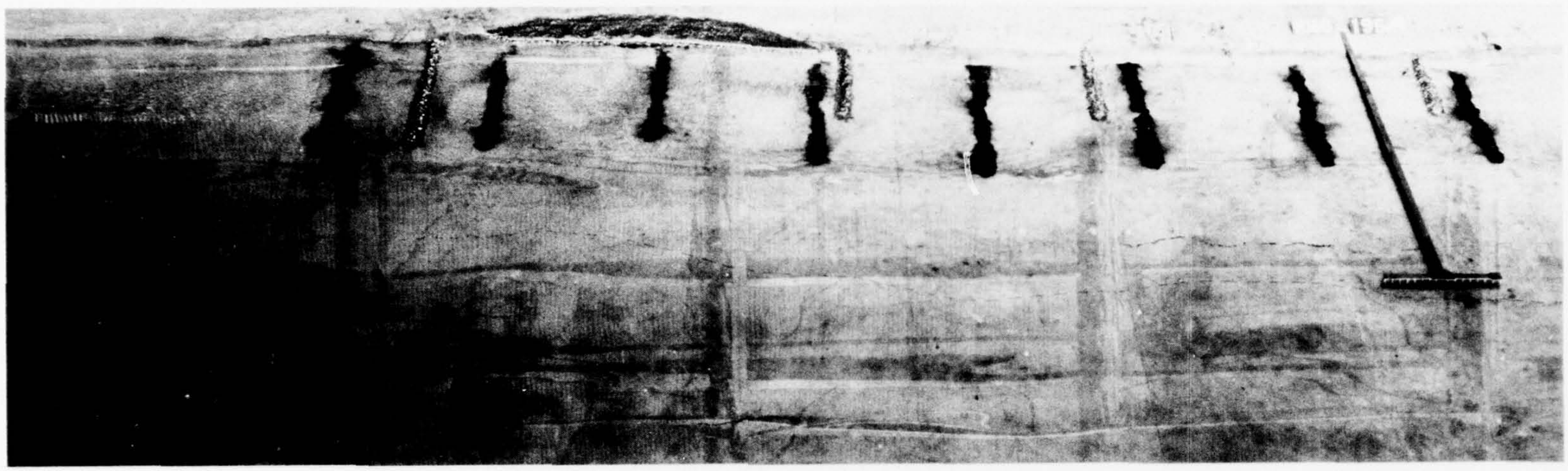


Photo 80. Typical wave and current patterns and current magnitudes (prototype feet per second) for the Authorized Groin *Plan*; 14-*sec*, 7-*ft waves from* S60°W at mhhw



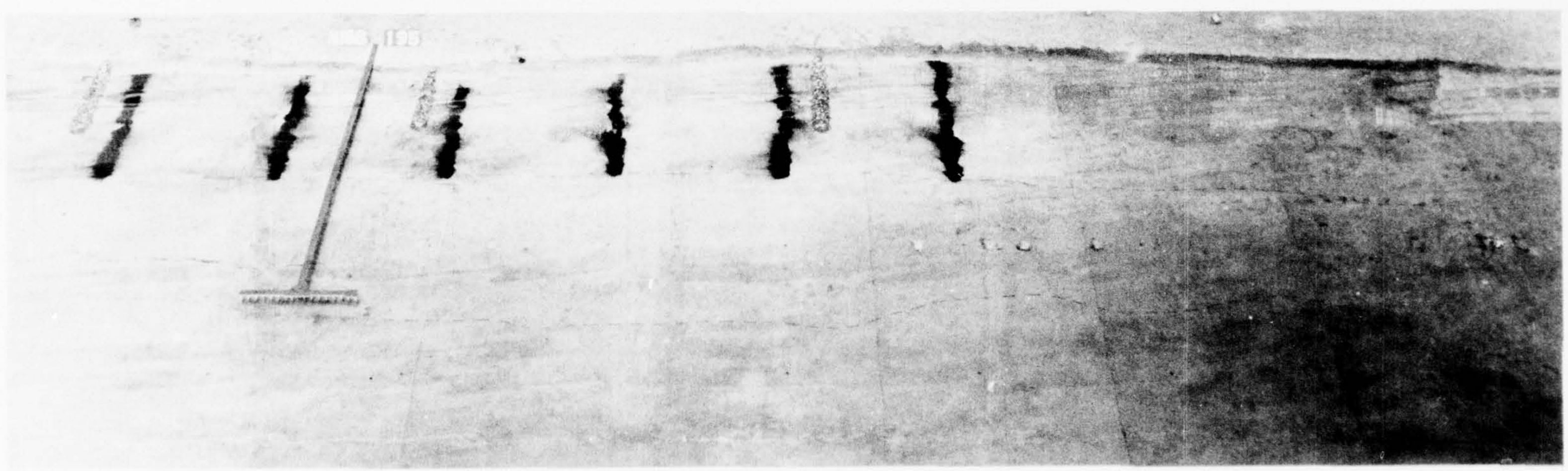


Photo 81. Tracer placement for the Authorized Groin Plan

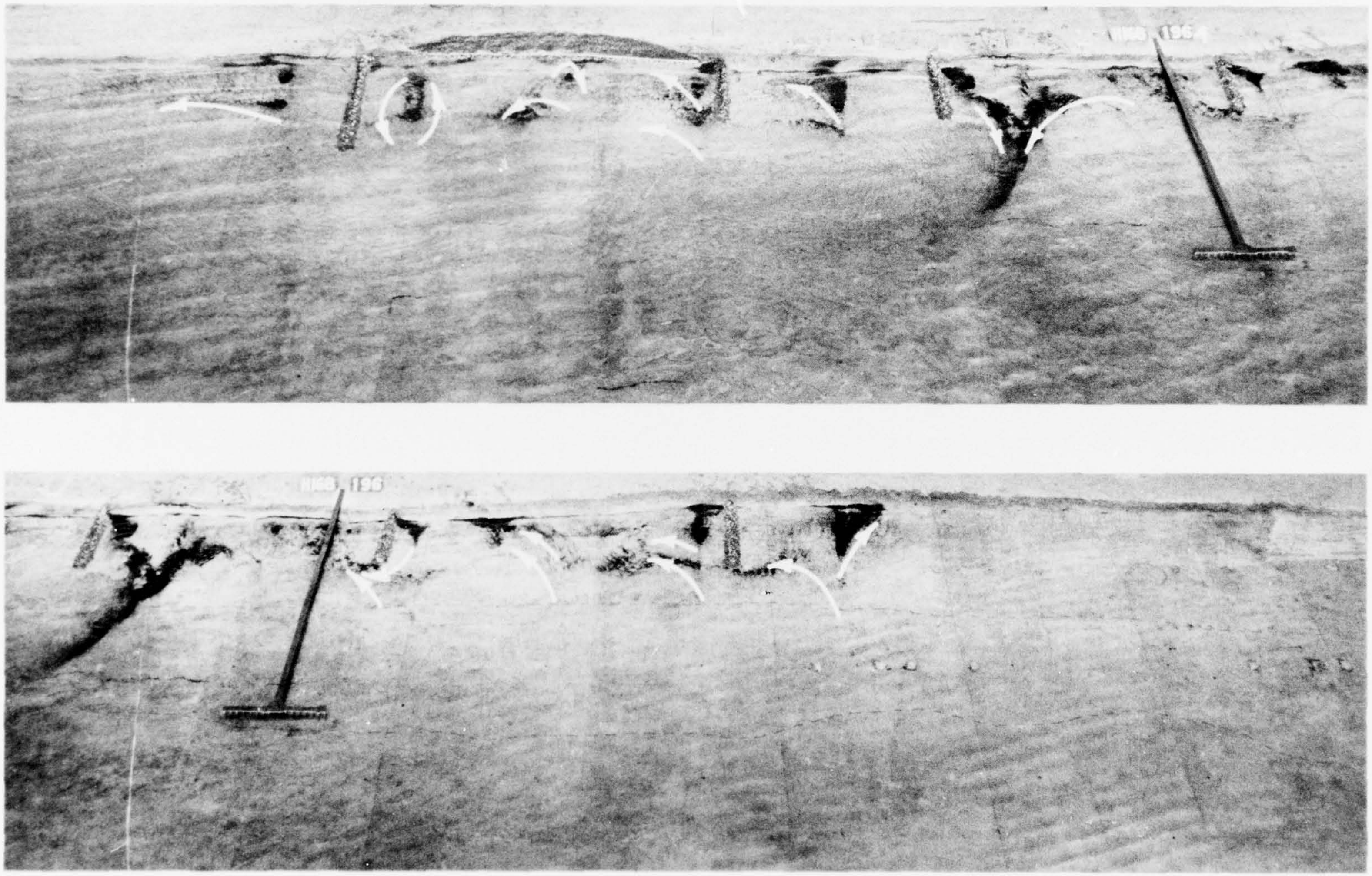


Photo 82. Typical tracer movement for the Authorized Groin Plan resulting from 14-sec, 7-ft waves from S60°W at mhhw

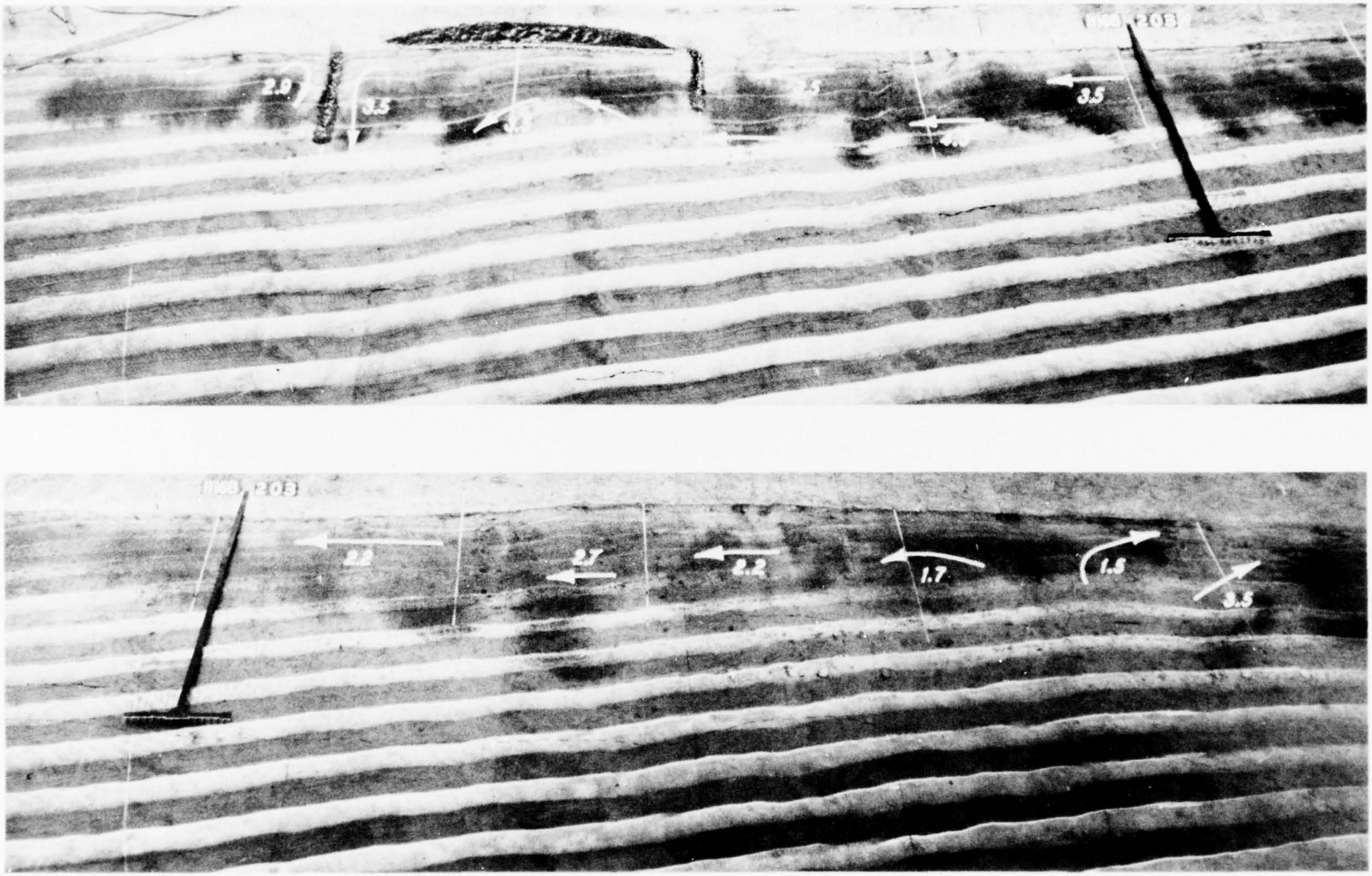


Photo 83. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 6; 7-sec, 11-ft waves from S60°W at mhhw

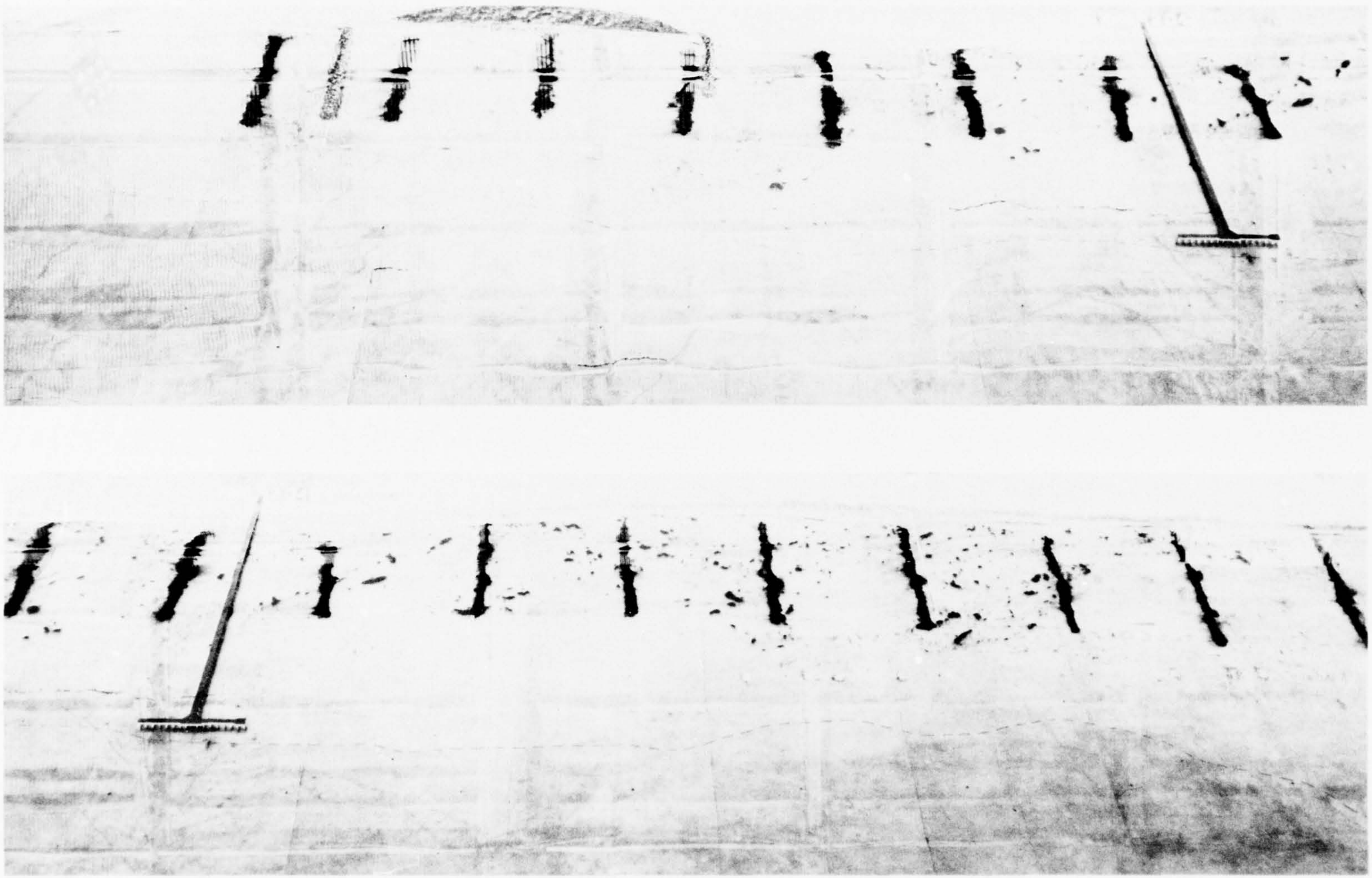


Photo 84. Tracer placement for Plan 6

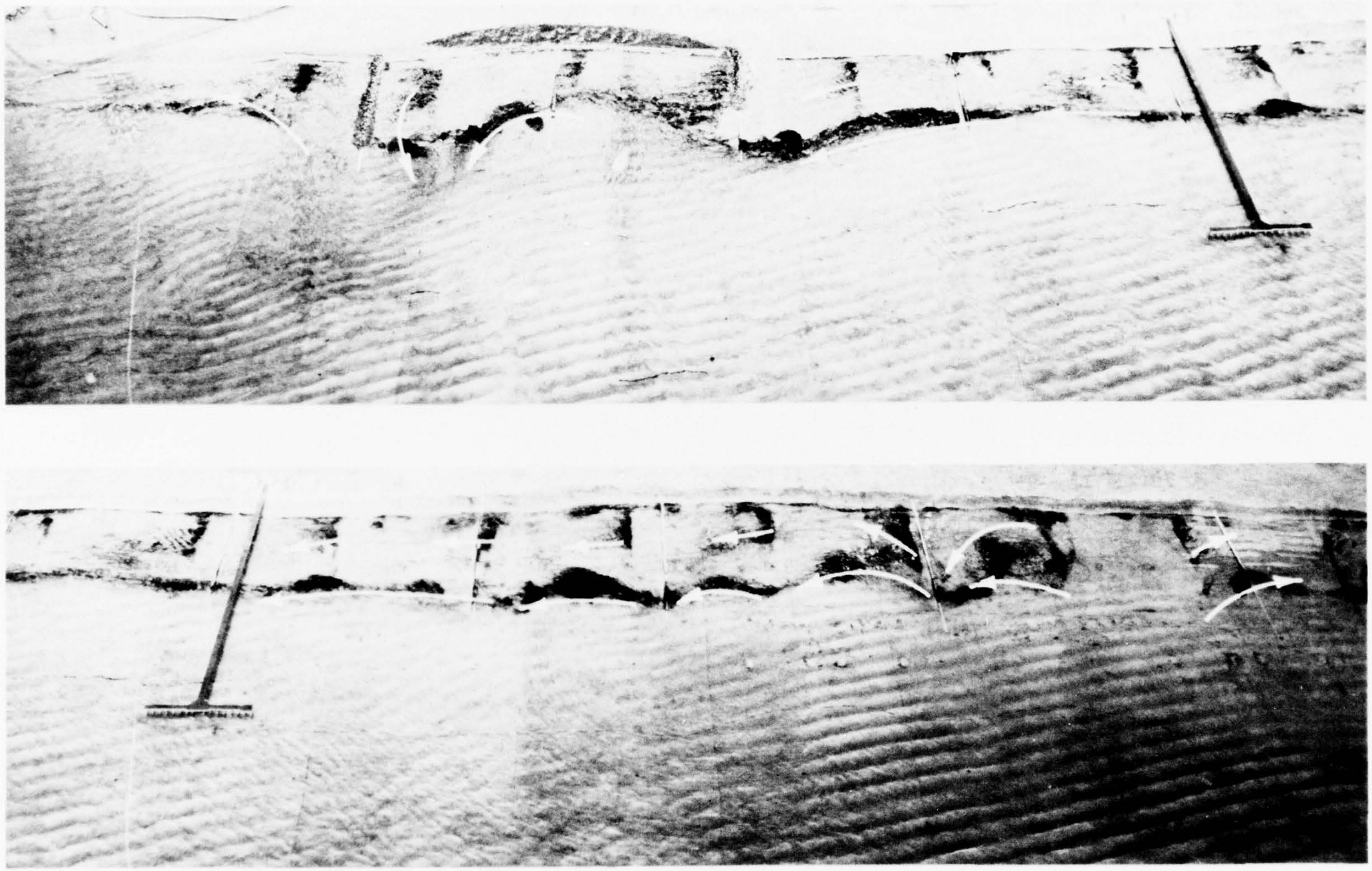


Photo 85. Typical tracer movement for Plan 6 resulting from 7-sec, 11-ft waves from S60°W at mhhw

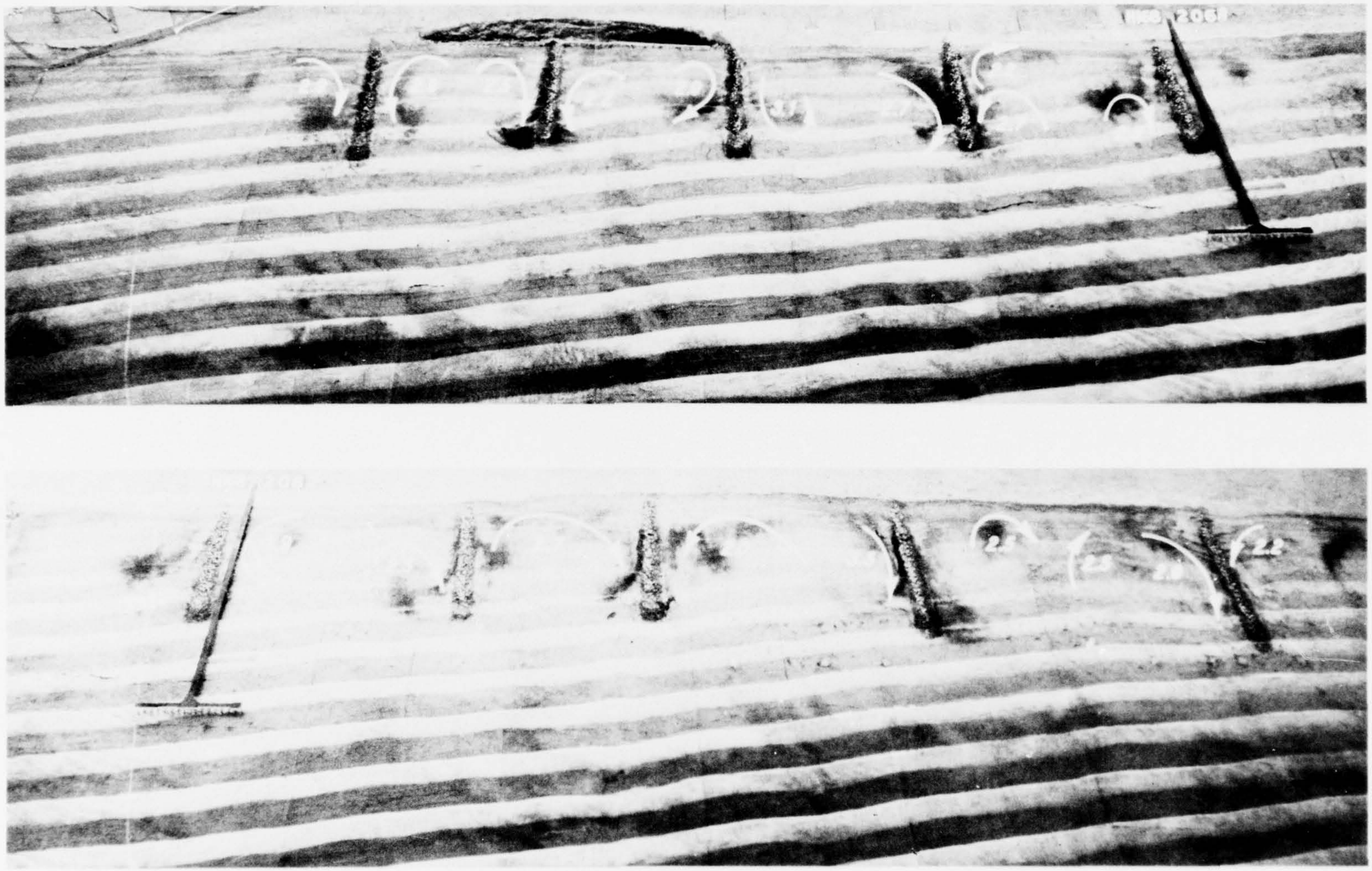


Photo 86. Typical wave and current patterns and current magnitudes (prototype feet per second) for Plan 7; 7-sec, 11-ft waves from S60°W at mhhw

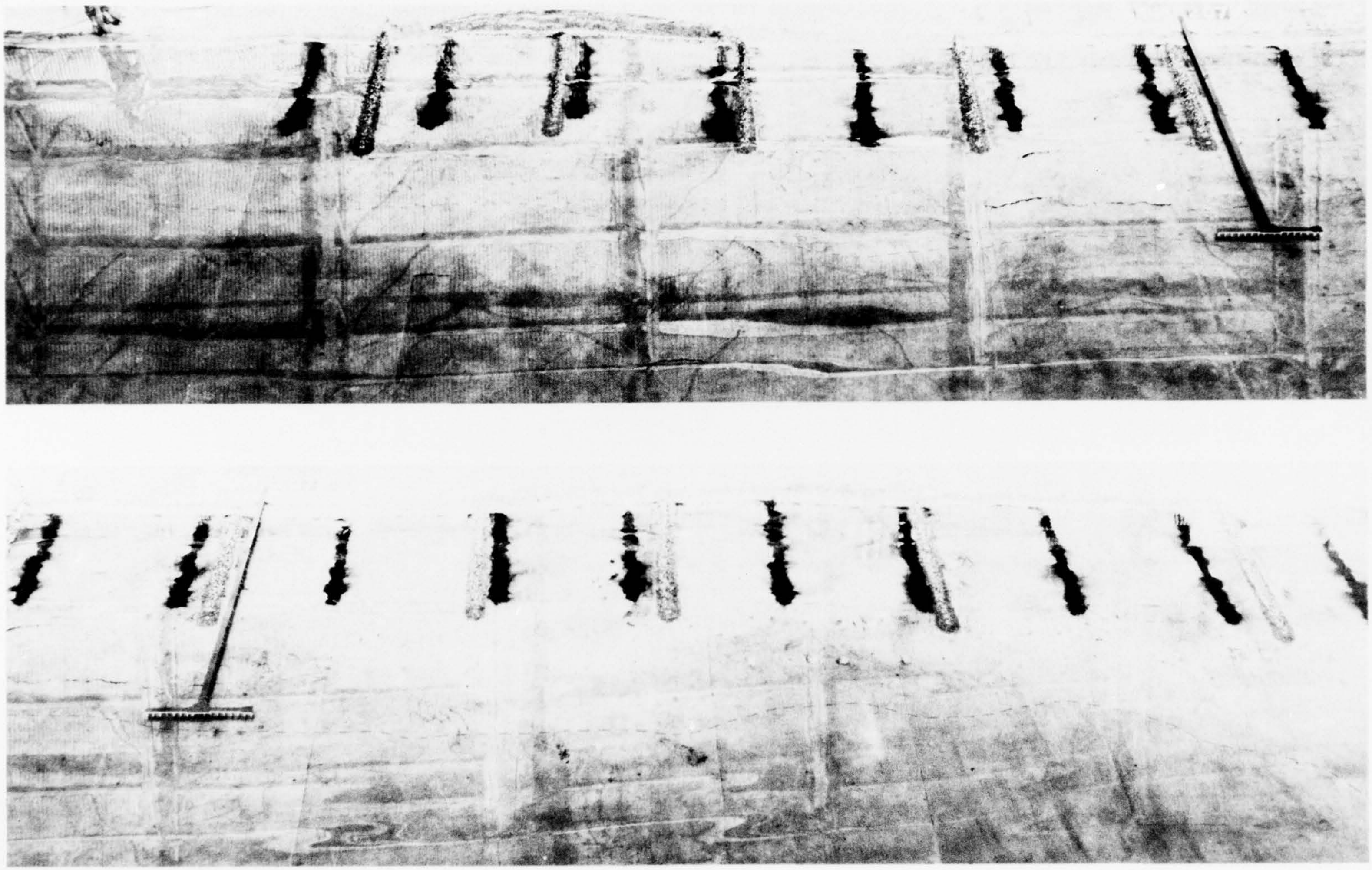
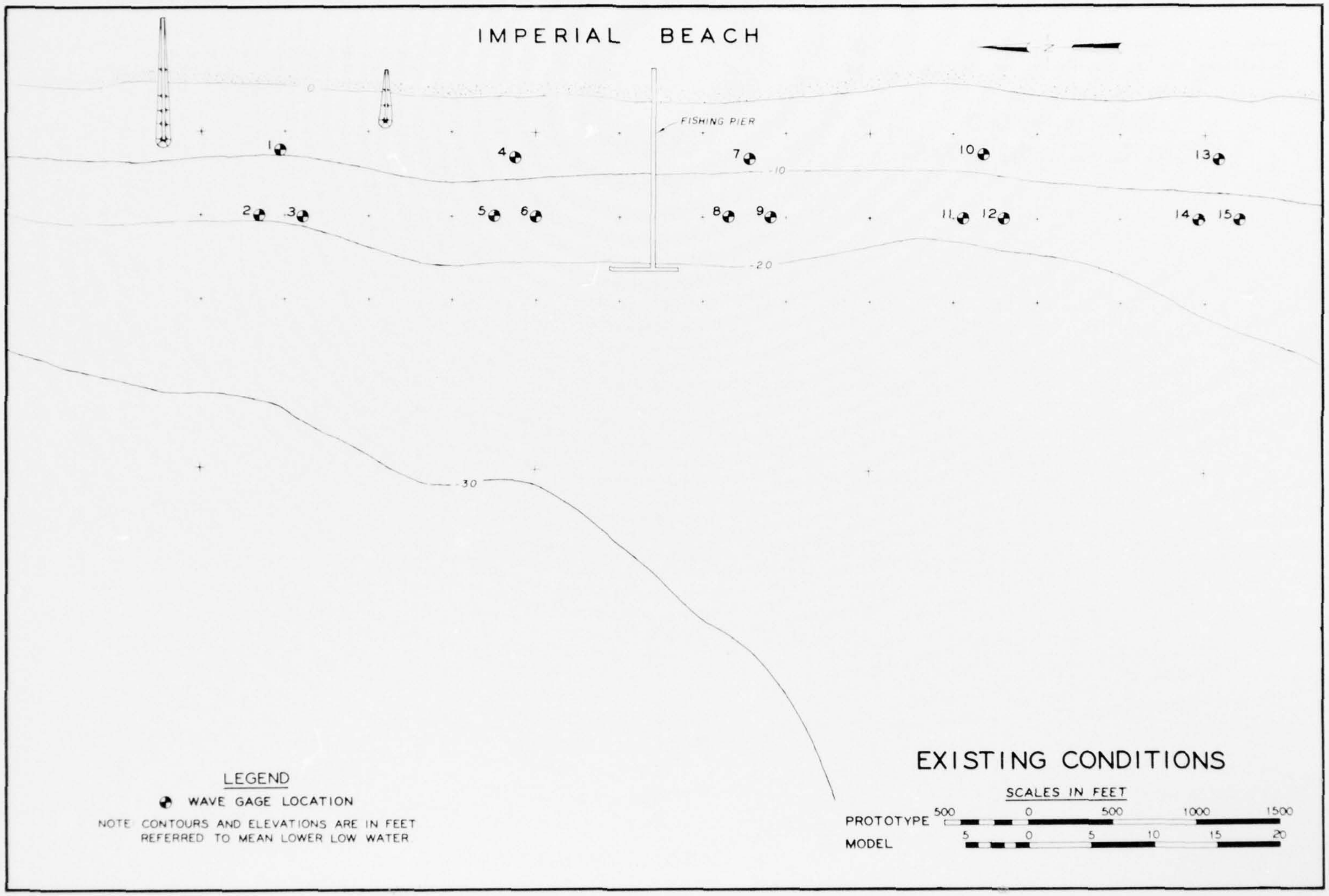


Photo 87. Tracer placement for Plan 7

Photo 88. Typical tracer movement for Plan 7 resulting from 7-sec, 11-ft waves from S60°W at mhhw



EXISTING CONDITIONS

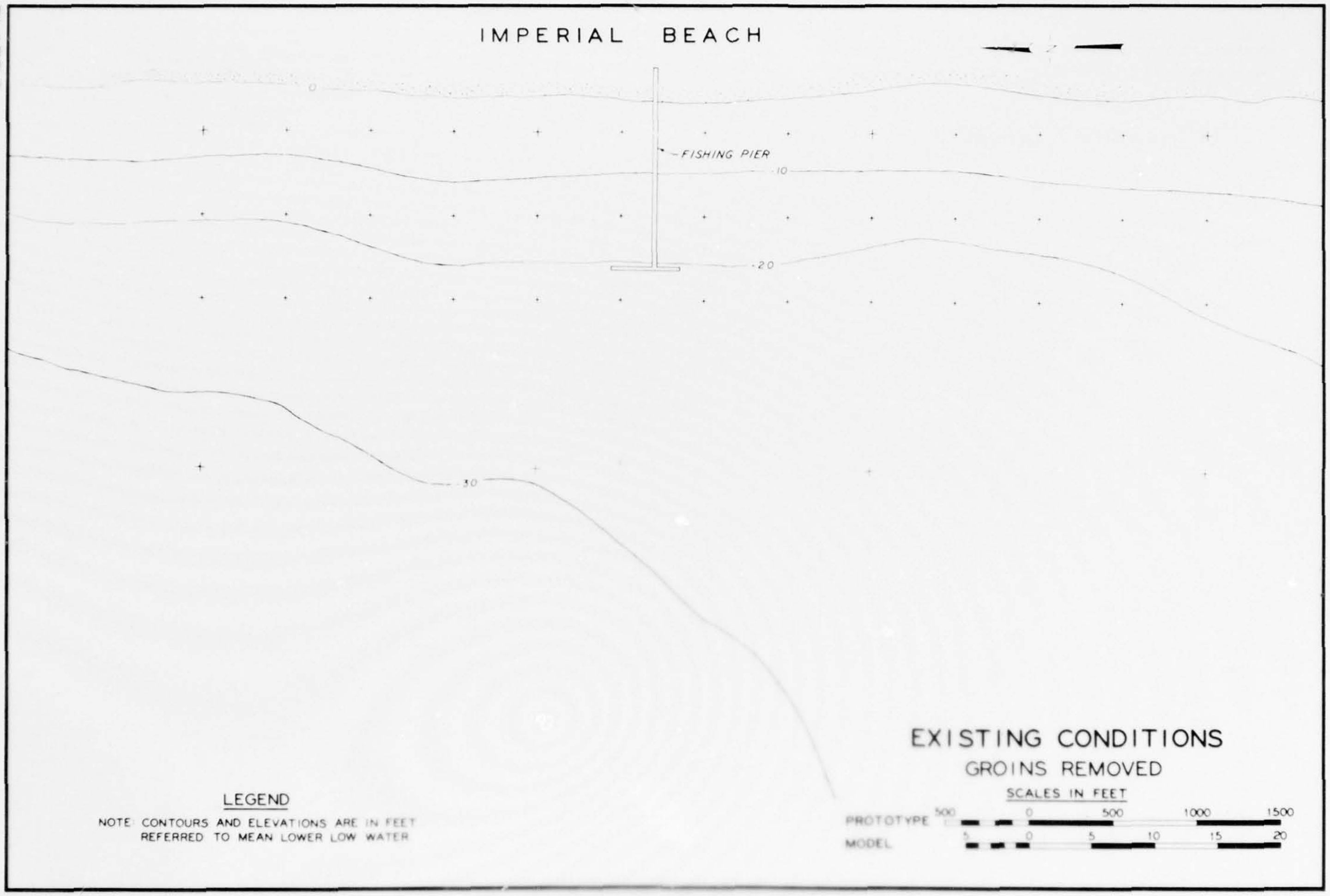
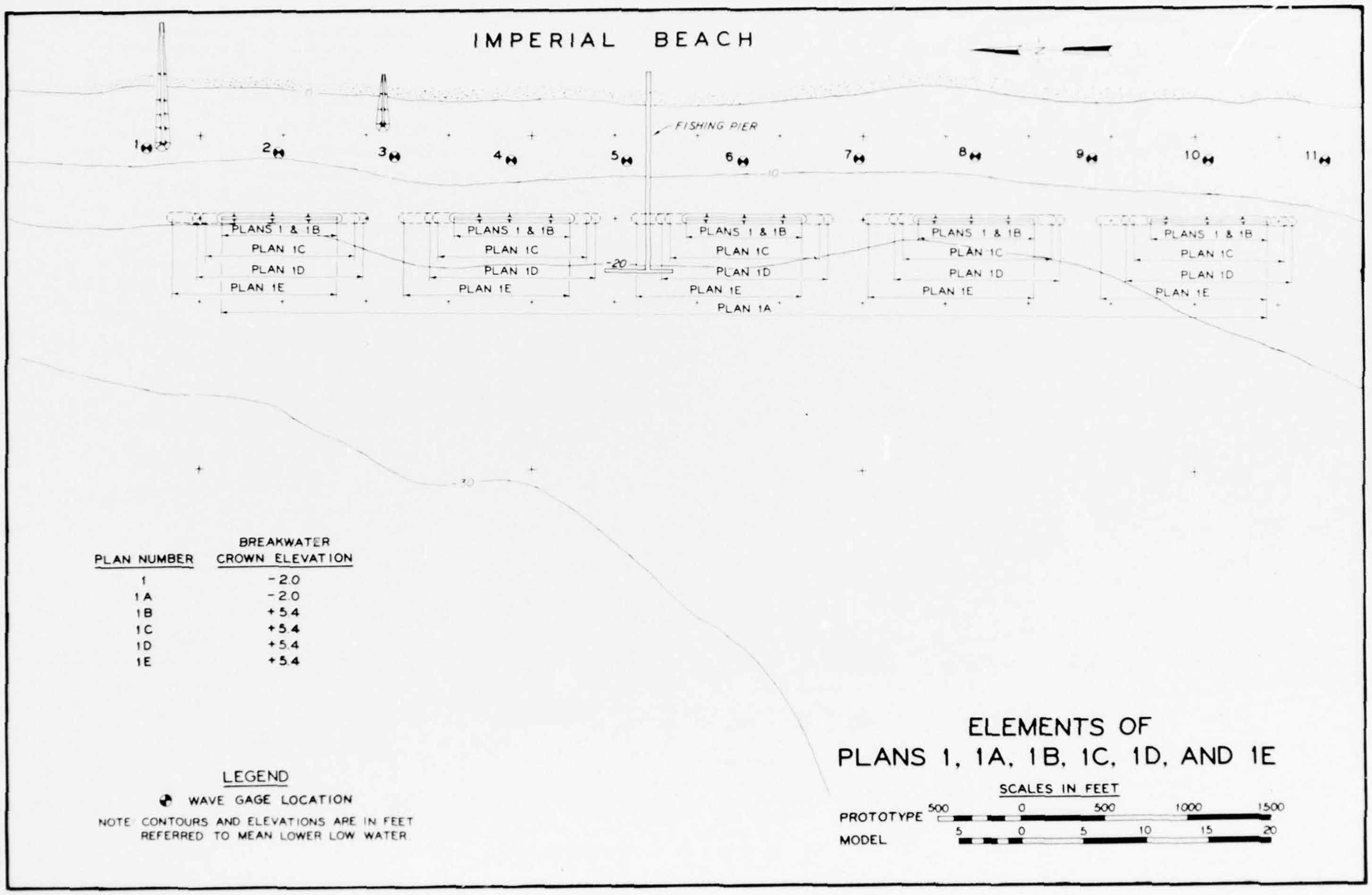

IMPERIAL BEACH
FISHING PIER
0
-10
-20
-30
EXISTING CONDITIONS
GROINS REMOVED
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15 20
LEGEND
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER

PLATE 2


IMPERIAL BEACH
FISHING PIER
1
2
3
4
5
6
7
8
9
10
11
-10
-20
-30
PLANS 1 & 1B
PLAN 1C
PLAN 1D
PLAN 1E
PLAN 1A
PLAN NUMBER
BREAKWATER CROWN ELEVATION
1 -2.0
1A -2.0
1B +5.4
1C +5.4
1D +5.4
1E +5.4
LEGEND
WAVE GAGE LOCATION
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER
ELEMENTS OF
PLANS 1, 1A, 1B, 1C, 1D, AND 1E
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15 20

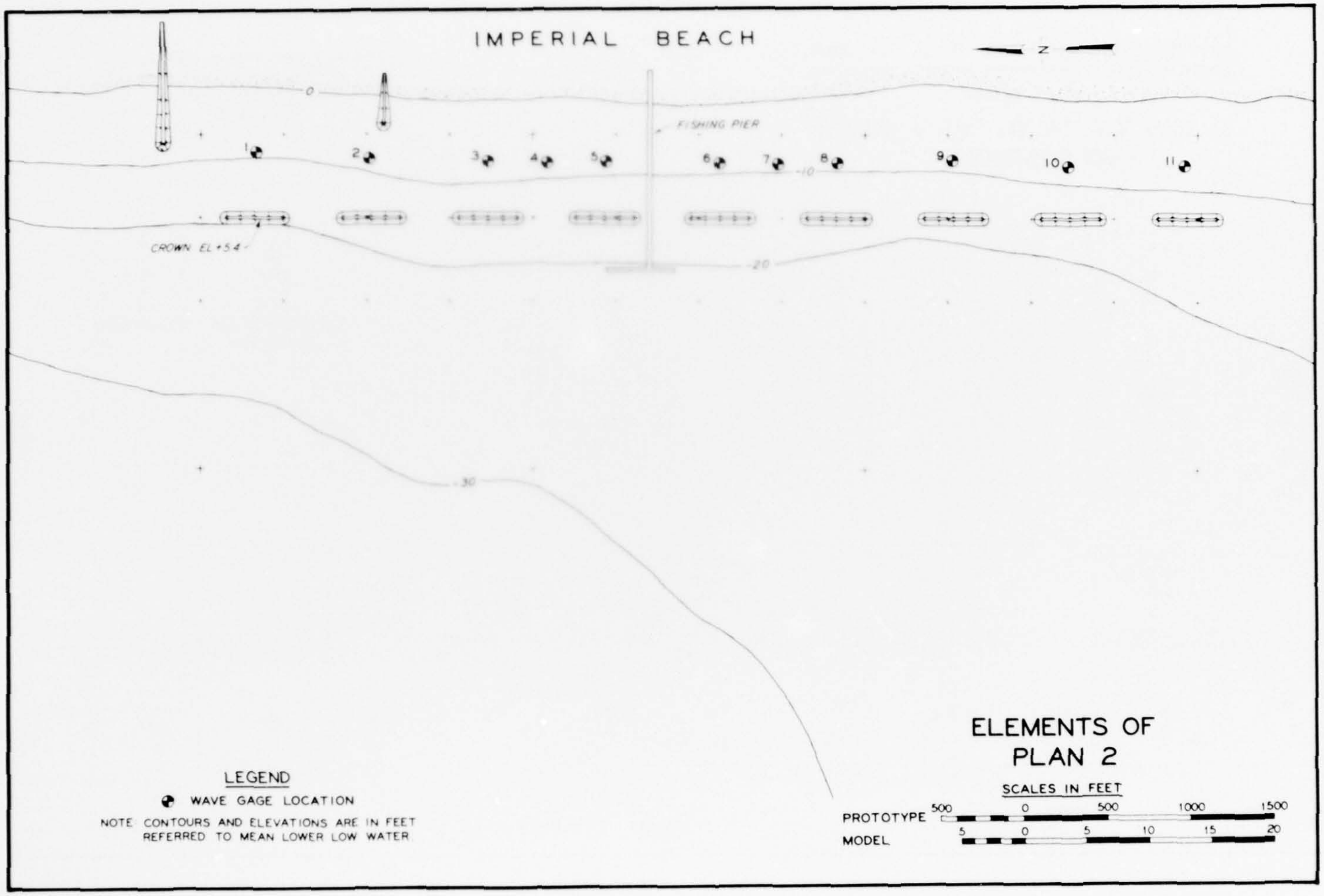

IMPERIAL BEACH
N
0
FISHING PIER
1
2
3
4
5
6
7
8
9
10
11
-10
CROWN EL +5.4
-20
-30
ELEMENTS OF
PLAN 2
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15 20
LEGEND
WAVE GAGE LOCATION
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET
REFERRED TO MEAN LOWER LOW WATER

PLATE 4

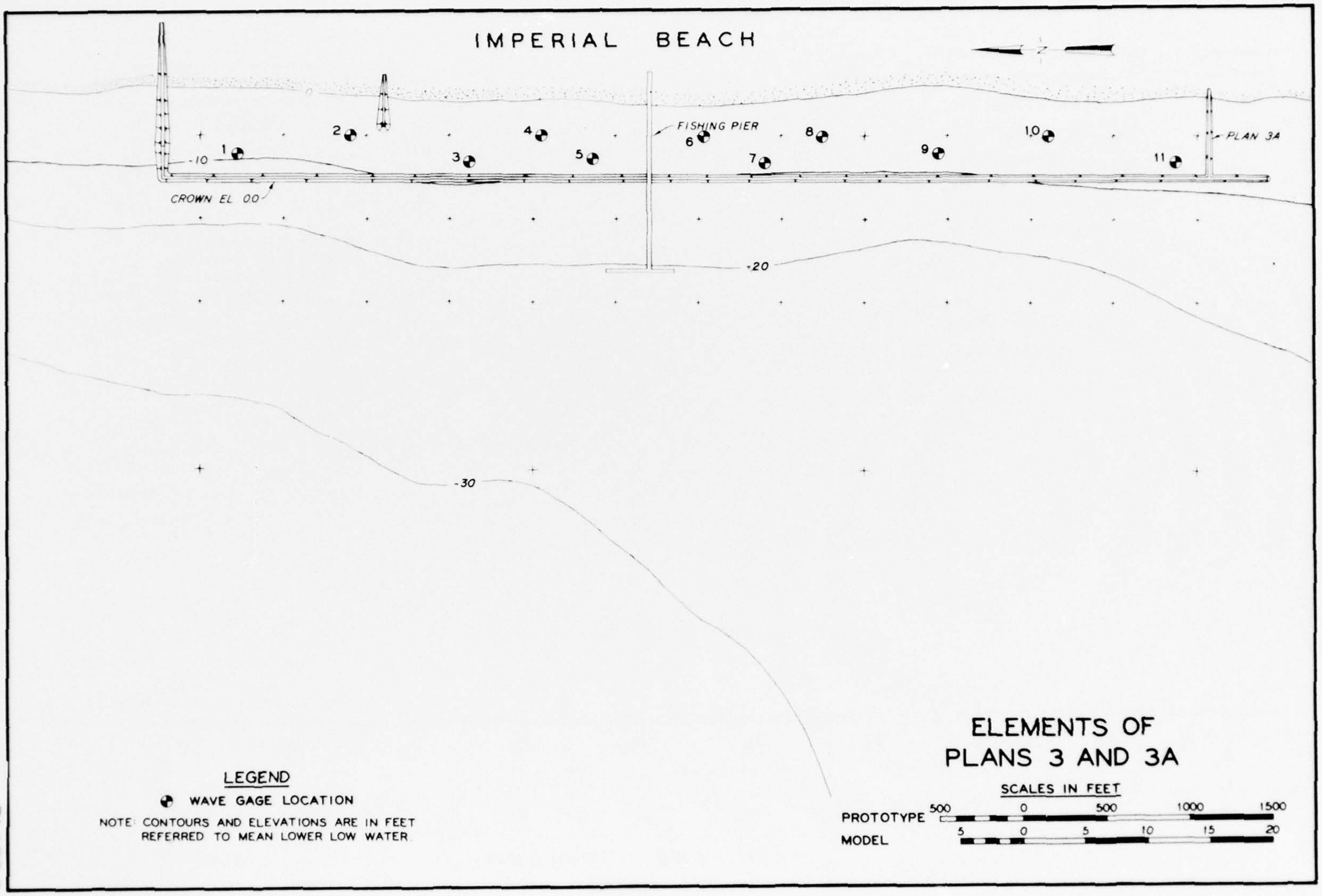

IMPERIAL BEACH
FISHING PIER
PLAN 3A
CROWN EL 00
-10
-20
-30
1
2
3
4
5
6
7
8
9
10
11
ELEMENTS OF
PLANS 3 AND 3A
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15 20
LEGEND
WAVE GAGE LOCATION
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET
REFERRED TO MEAN LOWER LOW WATER

PLATE 5

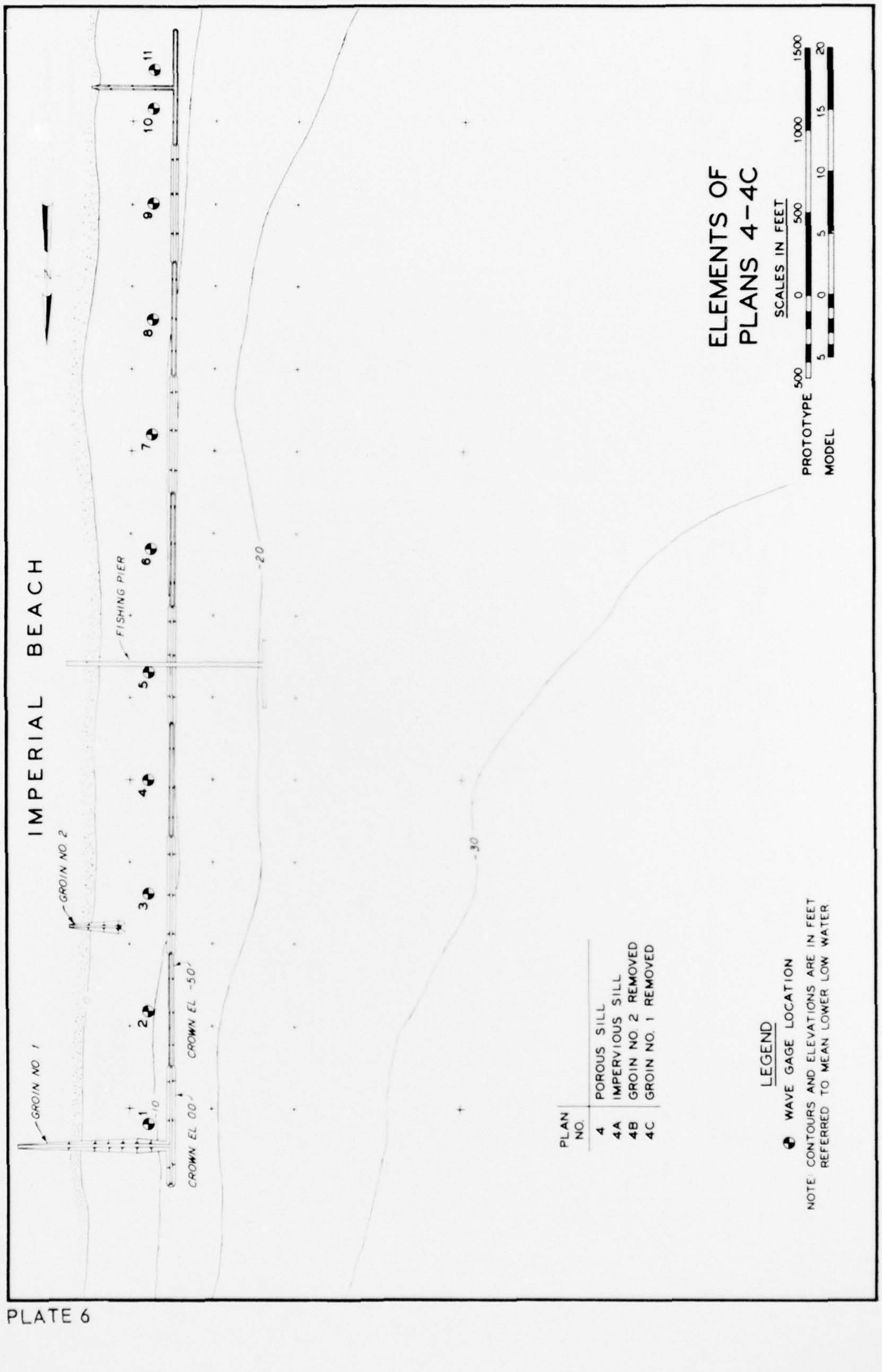
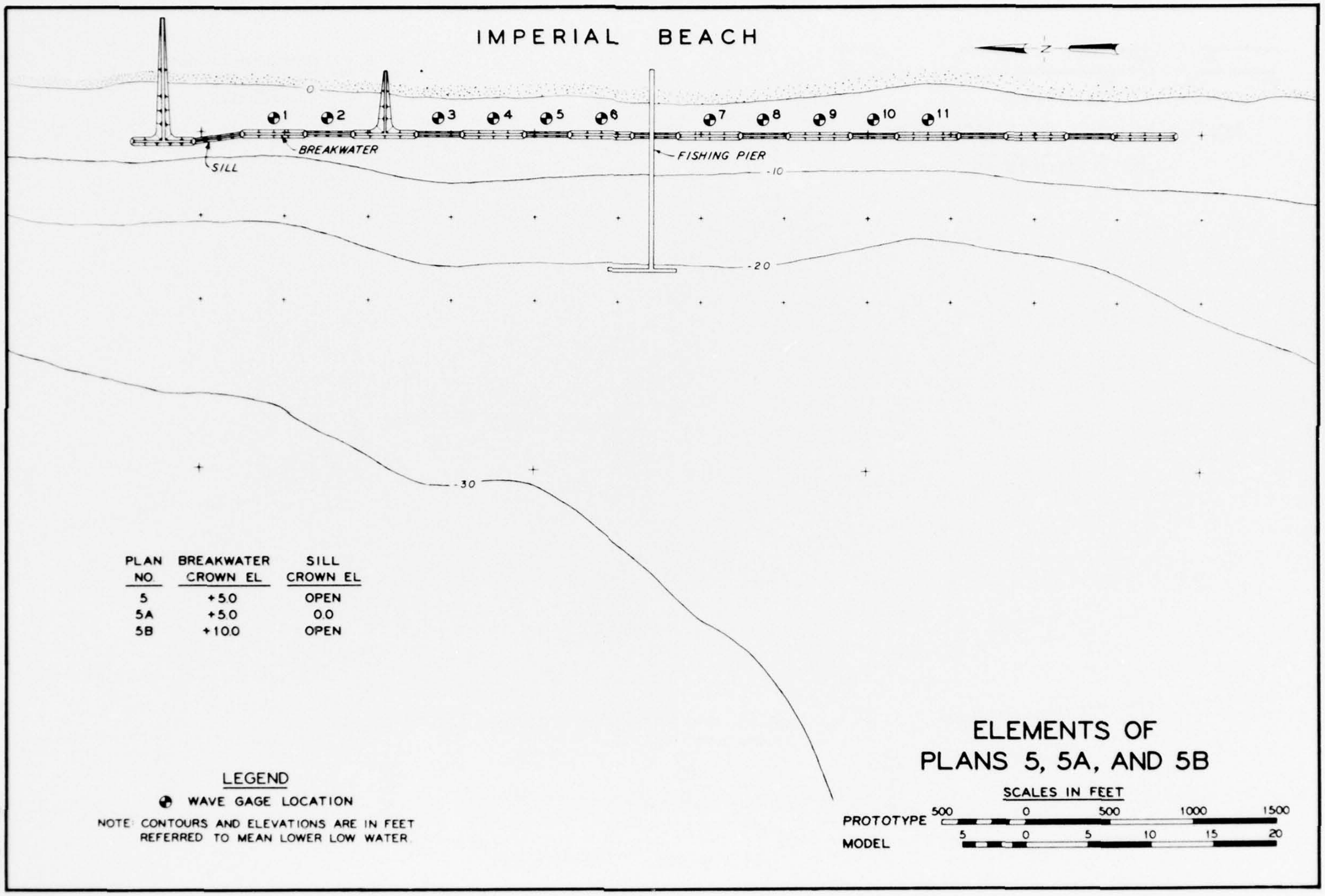

IMPERIAL BEACH
GROIN NO 1
GROIN NO 2
FISHING PIER
CROWN EL 0.0'
CROWN EL -5.0'
-10
-20
-30
1
2
3
4
5
6
7
8
9
10
11
N
ELEMENTS OF
PLANS 4-4C
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15 20
PLAN NO.
4 POROUS SILL
4A IMPERVIOUS SILL
4B GROIN NO 2 REMOVED
4C GROIN NO. 1 REMOVED
LEGEND
WAVE GAGE LOCATION
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET
REFERRED TO MEAN LOWER LOW WATER

PLATE 6



PLAN NO.	BREAKWATER CROWN EL	SILL CROWN EL
5	+5.0	OPEN
5A	+5.0	0.0
5B	+10.0	OPEN

LEGEND

WAVE GAGE LOCATION

NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER.

ELEMENTS OF PLANS 5, 5A, AND 5B

SCALES IN FEET

PROTOTYPE 500 0 500 1000 1500

MODEL 5 0 5 10 15 20

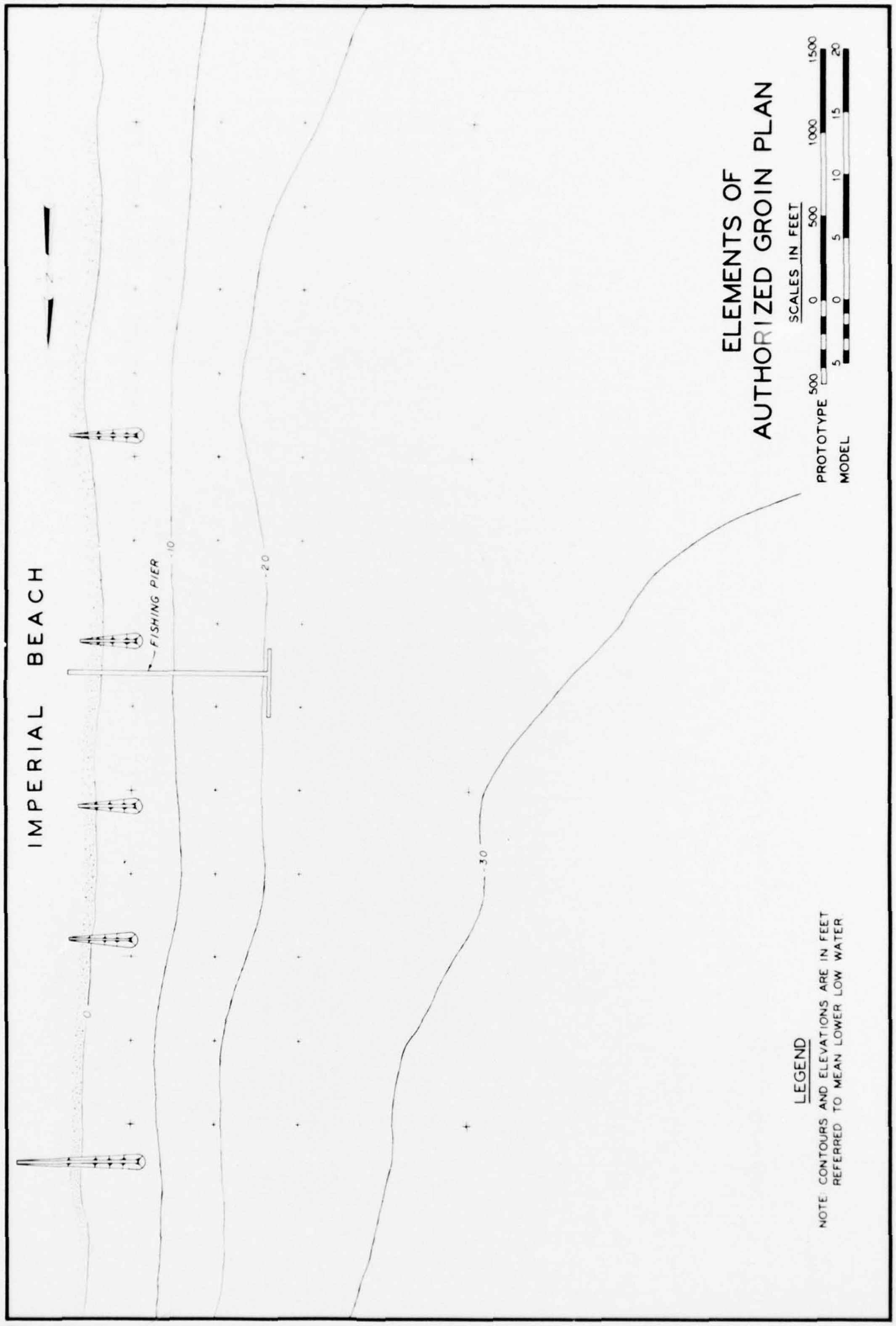

IMPERIAL BEACH
N
FISHING PIER
0
-10
-20
-30
ELEMENTS OF
AUTHORIZED GROIN PLAN
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15 20
LEGEND
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET
REFERRED TO MEAN LOWER LOW WATER

PLATE 8

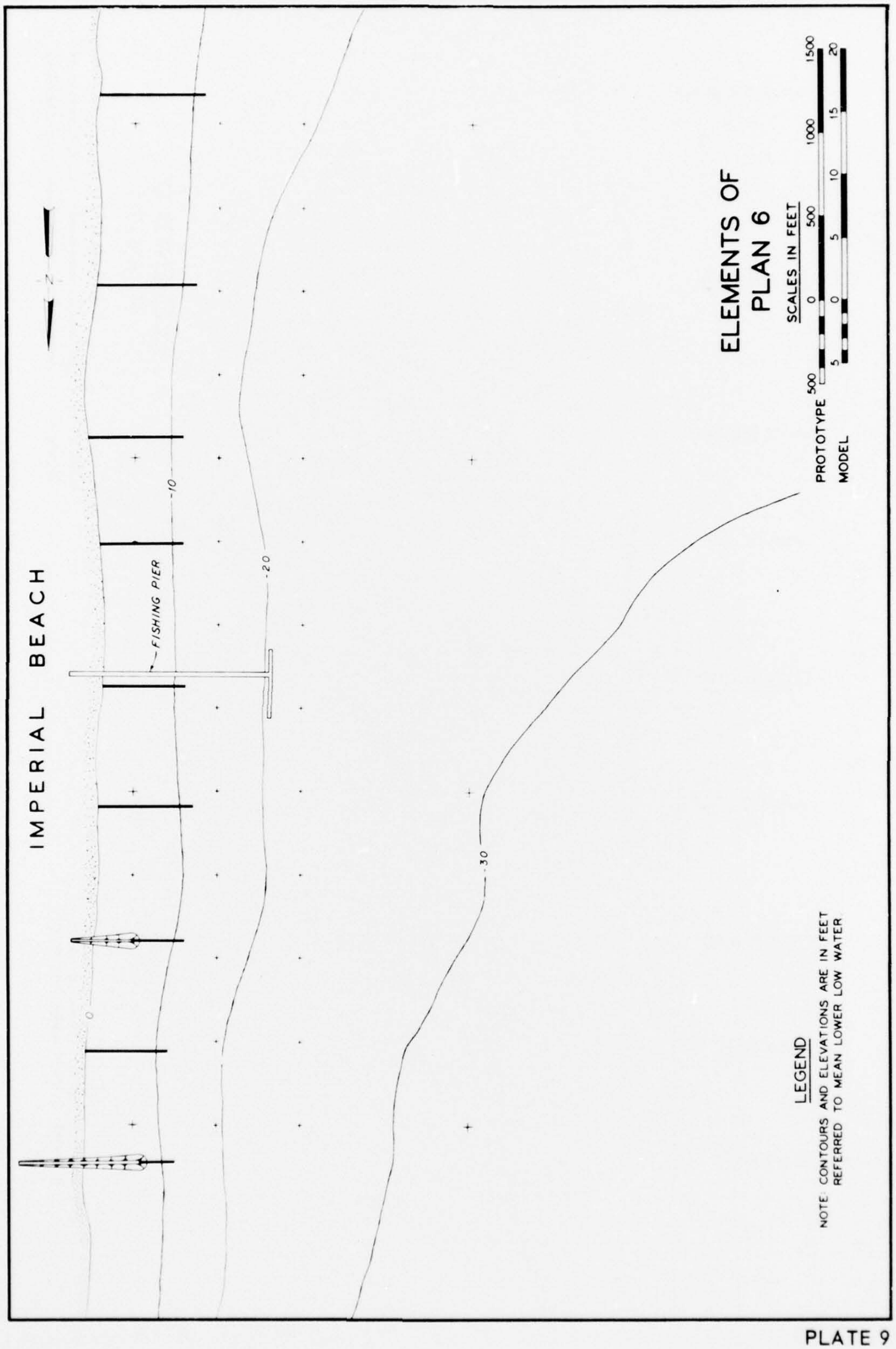

IMPERIAL BEACH
N
FISHING PIER
0
-10
-20
-30
ELEMENTS OF PLAN 6
SCALES IN FEET
PROTOTYPE 500 0 500 1000 1500
MODEL 5 0 5 10 15 20
LEGEND
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET REFERRED TO MEAN LOWER LOW WATER

PLATE 9

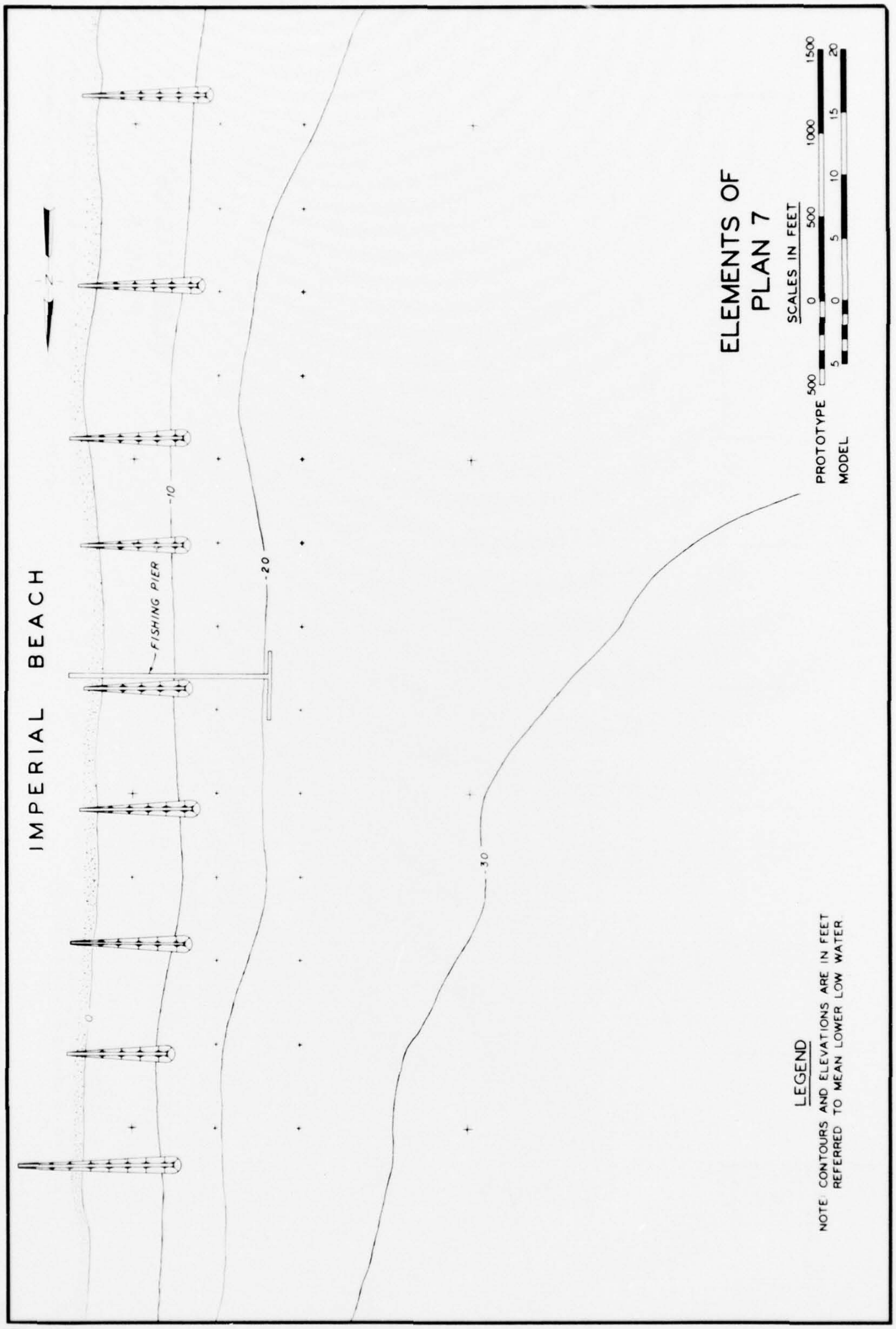
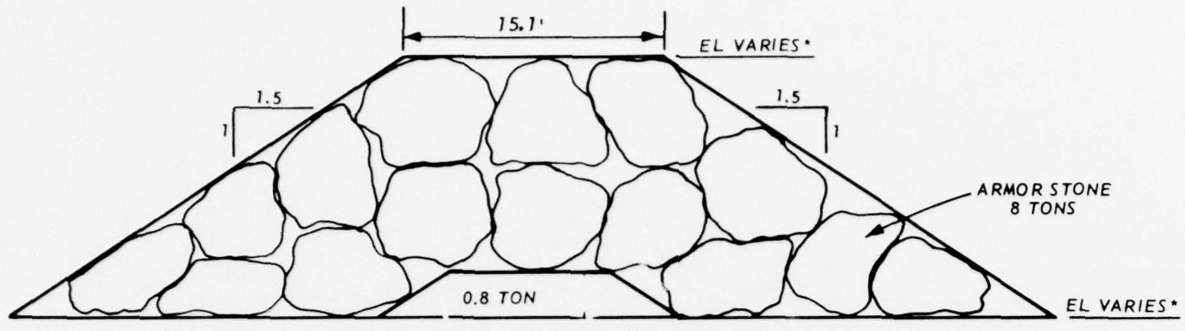
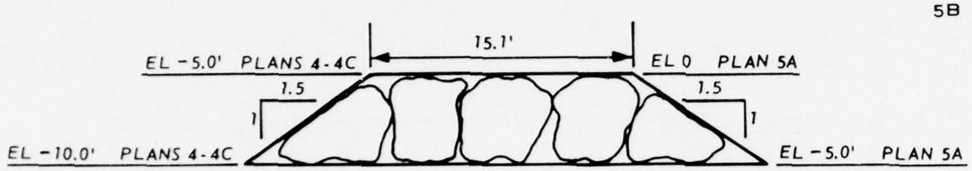

IMPERIAL BEACH
FISHING PIER
N
0
-10
-20
-30
ELEMENTS OF
PLAN 7
SCALES IN FEET
PROTOTYPE
500 0 500 1000 1500
MODEL
5 0 5 10 15 20
LEGEND
NOTE: CONTOURS AND ELEVATIONS ARE IN FEET
REFERRED TO MEAN LOWER LOW WATER

PLATE 10



BREAKWATER

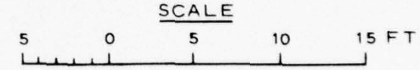
* PLAN	CREST ELEVATION, FT	TOE ELEVATION, FT
1 -1A	-2.0	-15.0
1B-2	+5.4	-15.0
3 -4C	0.0	-10.0
5 -5A	+5.0	-5.0
5B	+10.0	-5.0

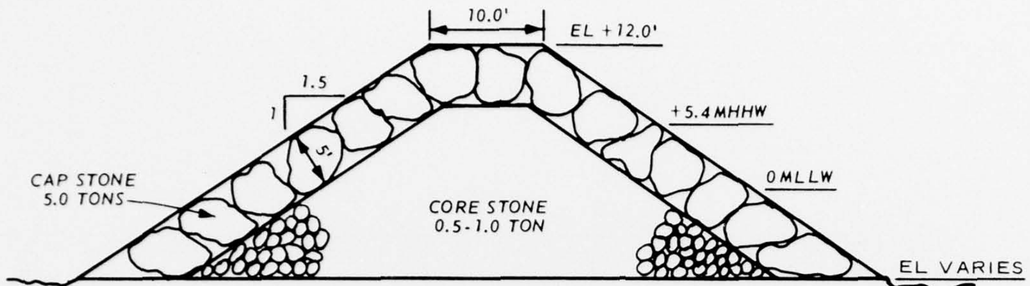


LOW-SILL

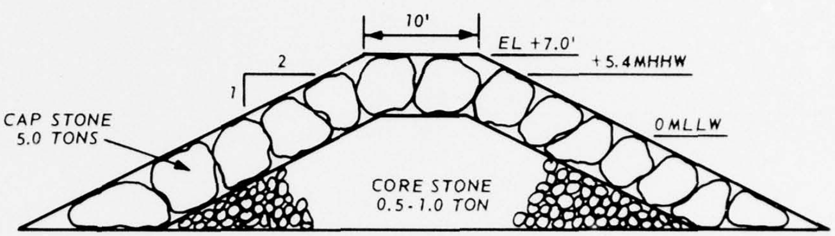
TYPICAL BREAKWATER AND LOW-SILL SECTIONS

SCALE





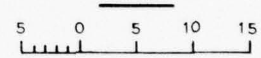
GROIN TRUNK



GROIN HEAD

GROIN SECTIONS
AUTHORIZED GROIN PLAN AND PLAN 7

SCALE



In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Curren, Charles R
Imperial Beach, California, design of structures for beach erosion control; hydraulic model investigation / by Charles R. Curren, Claude E. Chatham, Jr. Vicksburg, Miss. : U. S. Waterways Experiment Station, 1977.
44, [100] p., 12 leaves of plates : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; H-77-15)
Prepared for U. S. Army Engineer District, Los Angeles, Los Angeles, California.
References: p. 43-44.

1. Beach erosion. 2. Breakwaters. 3. Erosion control. 4. Groins (Structures). 5. Hydraulic models. 6. Water wave experiments. I. Chatham, Claude E., joint author. II. United States. Army. Corps of Engineers. Los Angeles District. III. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; H-77-15.
TA7.W34 no.H-77-15

LMED

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